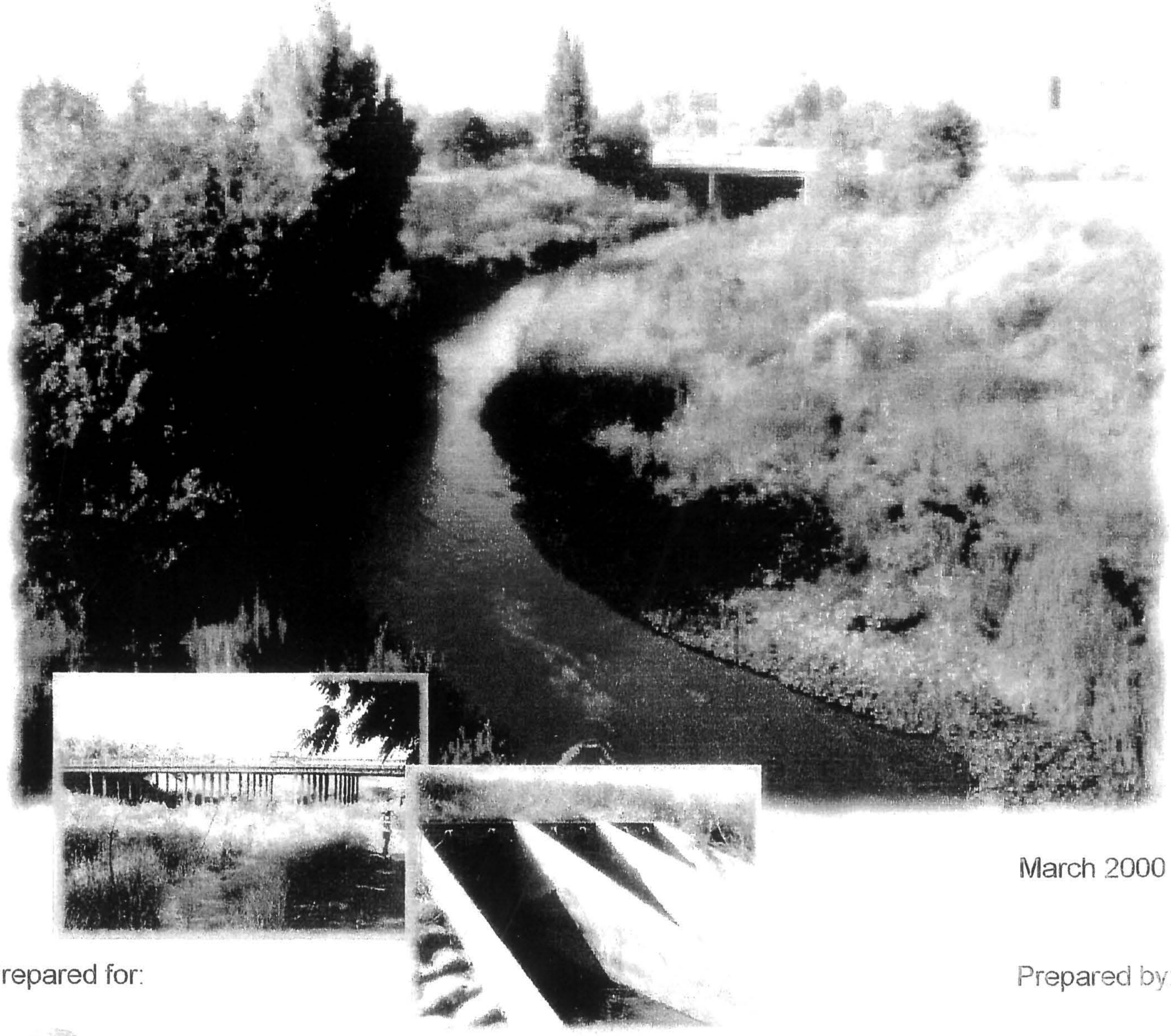
EXISTING CONDITIONS REPORT

HISTORY OF BASIN AND CHANNEL AND BASELINE HYDRAULIC EVALUATION

Santa Clara Valley Water District



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EXECUTIVE SUMMARY

The present Lower Guadalupe River channel cannot contain the design flood. If improvements are not made, major floods will overtop existing levees and inundate developed floodplain areas. The Santa Clara Valley Water District (SCVWD or Water District) is responsible for flood protection along the Lower Guadalupe River. In June 1999 the Water District retained a consultant team led by CH2M Hill to evaluate the capacity of the existing river channel and develop alternatives to prevent flooding during the design event. The study reach extends upstream from the UPRR (formerly SPRR) bridge near the community of Alviso to the Interstate 880 bridge near the south end of the San Jose airport, a distance of approximately 10,600 meters. Historically, portions of this reach have been straightened, enlarged, and confined between earthen levees to prevent flooding of valuable floodplain properties.

This report describes the results of the baseline hydraulic study. Results from the baseline study provide background information required to identify and understand factors contributing to the limited capacity of the existing channel. Understanding this information is required before appropriate alternatives can be developed to increase the system's capacity and satisfy the objectives of the design and ultimate project maintenance requirements. Northwest Hydraulic Consultants (NHC), a subconsultant to CH2M Hill, completed the baseline hydraulic study and prepared this report. The report is divided into two parts. Part 1 discusses the history of the river basin and channel, while Part 2 describes NHC's approach and results from a water surface profile computer model created to examine the capacity of the existing channel and evaluate the performance of proposed flood protection alternatives.

Information presented in Part 1 is taken primarily from previous investigations and published literature. In January 2000, NHC completed a comprehensive sediment engineering investigation for the Lower Guadalupe River between San Francisco Bay and Interstate 880. The study was completed for the Water District under a contract separate from this project. The sediment study report contains a thorough review of the geomorphic history of the basin and channel and describes the primary factors that have influenced channel capacity. These include both natural and man-induced factors such as: land subsidence due to groundwater withdrawal, rising sea levels, changes in basin land use, sediment accumulation, bridge constructions, and aggressive growth of in-channel vegetation. All of these contribute to decreasing channel capacity with time and to increasing the annual Water District maintenance requirements.

Part 2 of the report describes NHC's approach for assessing baseline hydraulic conditions, summarizes the design flood flow rates, and discusses the development of a HEC-RAS water surface profile model of the project reach. The design flood flow rates were prescribed by the Water District and are described in Directive No. 1 in Appendix A. At the upstream end of the study reach, at Interstate 880, the design discharge corresponds to the U.S. Army Corps of Engineers published 100-yr flow rate of 17,000 cfs (481.4 cms). This is the same discharge used by the Corps to design the Downtown Guadalupe River flood protection project which begins at Interstate 880 and extends upstream to Interstate 280. In Directive No. 1, the Water District increases the design discharge incrementally downstream to account for flow contributions from storm water pump stations. At the downstream end of the project reach -- the UPRR bridge -- the design discharge is 20,000 cfs.

An "Existing Conditions" HEC-RAS model was created from an existing HEC-2 model of the project reach. The HEC-2 model, developed by NHC for the earlier sediment study, was itself developed from a series of sub-reach HEC-2 models initially constructed by the Water District. The channel geometry in these models is based upon 1996 topographic maps and field surveys prepared by the Water District. NHC calibrated the HEC-RAS model to reproduce flood levels observed during the March 1995 flood of record. This calibration was verified by reproducing a January 1995 flood profile. The calibrated model should only be used for floods that exceed about 8,000 cfs. Below this, the model is likely to underestimate water levels because the influence of vegetation resistance increases significantly as flow depth decreases.

The HEC-RAS model confirmed that the existing channel cannot contain the design flood. Design water levels overtop the levees between the UPRR bridge and Trimble Road. Between Trimble Road and Highway 101, design water levels are lower than the levees although not by enough to satisfy freeboard requirements. Upstream from Highway 101, the amount of freeboard continues to increase. At the downstream side of Interstate 880, the predicted design water surface elevation is lower than a target elevation of 17.86 meters required by the Water District (see Directive No. 5, Appendix A). Pressure flow conditions exist during the design event at all bridges from Gold Street through Highway 101, inclusive. Overtopping conditions exist at two bridges: upstream (east bound) Highway 237 and Montague Expressway. The baseline HEC-RAS model includes the U.S. Army Corps of Engineers SRA plantings, but does not include the Caltrans Interstate 87 mitigation (as originally specified in Directive No. 3, see Appendix A).

The HEC-RAS model quantifies the overall capacity of channel sub-reaches, without regard to freeboard or to future changes in vegetation, roughness, or sediment accumulation. Between the UPRR bridge and Highway 237, incipient overtopping (capacity) of the existing levees corresponds to approximately 17,000 cfs (481.4 m^3/s). Above Highway 237 and upstream to Montague Expressway, the channel capacity drops to approximately 14,000 cfs (396.4 m^3/s), although there are some sections in this reach capable of passing up to approximately 15,500 cfs (438.9 m^3/s). Channel capacity gradually increases from Montague upstream to Highway 101. Just above Montague the capacity is about 15,000 cfs (424.8 m^3/s), at Trimble about 20,000 cfs (566.3 m^3/s), and below Highway 101 exceeding 23,000 cfs (651.3 m^3/s). Just above Highway 101 the channel capacity drops to about 18,500 cfs (523.9 m^3/s), due mainly to head losses through the Highway 101 bridge, then increases with distance upstream to about 23,000 cfs (651.3 m^3/s) at Airport Parkway. Capacity upstream of there to Interstate 880 exceeds 23,000 cfs.

The results presented herein are based on modeling calibrated to the March 1995 event. Because of possible changes that may have occurred to vegetation, roughness, and sediment accumulation since 1995, NHC strongly recommends high water surface profile data be collected during the year 2000 runoff season. It is recommended that high peak profiles as well as both intermediate (4,000 to 8,000 cfs) and low (less than 2,000 cfs) profile data be collected. This will allow development of dependable model parameters for the full range of expected flow conditions (from 2-year to 100-year).

INTRODUCTION

Flood protection improvements are needed along the Lower Guadalupe River to safely convey the design discharge through the northern sections of the Cities of San Jose and Santa Clara, California. Located between San Francisco Bay and downtown San Jose, the Lower Guadalupe River receives runoff from a 160-square mile drainage basin comprised of a steep upper watershed and a highly urbanized valley floor (Figure 1). This study examines the reach of the Lower Guadalupe River that extends upstream from the UPRR (formerly SPRR) bridge near the community of Alviso to the Interstate 880 bridge near the south end of the San Jose airport, a distance of approximately 10,600 meters (Figure 2). This stretch of the river will be referred to as the project reach. Presently, flood protection is provided along the project reach continuous parallel earthen levees. However, for the current channel condition, the levees are generally too low and will be overtopped during the design discharge. The levees were originally built in 1963, but sections were raised in the 1980s and 1990s. No other major flood protection features are present along the project reach.

Previous studies by and for the Water District have revealed that the existing flood control levees are too low to safely convey the design discharge for present channel conditions. A recent study by NHC for the Water District confirmed the lack of channel capacity and identified the following as the key physical features and processes that affect flood conveyance: local geology, regional seismology, tidal processes, subsidence, a rising sea level, changes in land use in the watershed and on the floodplains, urbanization, channel improvement projects upstream of Interstate 880, sediment accumulation from tidal and river processes, bridge constrictions and aggressive encroachment of in-channel vegetation (NHC, 2000).

The Water District is responsible for providing flood protection along the project reach. On June 30, 1999 the Water District retained CH2M Hill and their team of consultant experts to develop a flood protection plan to convey the design discharge through the project reach. The CH2M Hill team was also authorized to develop and evaluate maintenance activities that will increase channel capacity in the interim period 2000-2001, before the preferred flood protection project is constructed in 2002. CH2M Hill was also retained to complete a reconnaissance level engineering investigation to identify potential sources of flooding in the community of Alviso and identify environmental impacts that Guadalupe River flood protection projects may have along Alviso Slough and the neighboring salt ponds. Alviso Slough extends downstream of the project reach and is a continuation of the Lower Guadalupe River and extends approximately 7,000 meters from San Francisco Bay to the UPRR bridge, the downstream end of the project reach.

This existing conditions report presents the results of the hydraulic baseline engineering study. Northwest Hydraulic Consultants (NHC) completed the study and prepared this report. The purpose of the study was to gain an understanding of the features and the processes that have led to the present channel condition. It is also to develop the numerical modeling tools needed to evaluate flood control alternatives.

The report is divided into two parts. Part 1 discusses the history of the basin and river as well as the processes that have influenced channel capacity. Part 2 describes the analytical approach and results from the development of the HEC-RAS water surface profile computer model that was created to examine the project reach and also discusses the existing capacity of the channel.

PART 1 – HISTORY OF BASIN AND CHANNEL

1.0 Sources of Data and Information

The information presented in Part 1 is taken mainly from two existing reports -- a sediment engineering study completed by NHC for the Santa Clara Valley Water District in January 2000 and a watershed planning study completed by the Water District in January 1997 (NHC 2000, SCVWD 1997). The NHC report describes the results of a detailed sediment engineering investigation of the Lower Guadalupe River between San Francisco Bay and Interstate 880. It contains a thorough description of the history of geologic and geomorphic processes that have affected the river's ability to convey flood water. The Water District's planning study for the Lower Guadalupe River contains useful information including descriptions of past flooding problems and maintenance activities.

In addition to these reports, the Water District, CH2M Hill, and the other team members provided a variety of information used for the baseline study. These included historic flood high water mark data, topographic maps, bridge plans, agency contacts, and much more. The Water District provided the CH2M Hill team with five written directives to provide direction on how to proceed with certain project activities. They are identified below and attached in Appendix A:

Directive #1 – Design Flow Rates

Directive #2 – Hydraulic Model Selection

Directive #3 – Projects to Include in Baseline Conditions

Directive #4 – Cumulative Environmental Effects Analysis for the Bay Lands Area Only

Directive #5 – Hydraulic Integration with Downtown Project

2.0 History of Geologic and Geomorphic Changes

2.1 Characteristics of the Watershed and Project Reach

Located in Santa Clara County, the Guadalupe River basin covers an area of approximately 160 square miles. The river originates in the Santa Cruz Mountains, located southwest of San Jose, and drains northly through the heavily populated Santa Clara Valley and into San Francisco Bay (Figure 1). Major tributaries of the Guadalupe River are Alamitos Creek, Canoas Creek, Ross Creek, and Los Gatos Creek. Basin relief is approximately 3,800 feet, with the highest point near Loma Prieta in the Santa Cruz Mountains and the lowest point near Alviso at San Francisco Bay.

Hydrology of the watershed has been greatly altered by human activity. The upper watershed is regulated by several reservoirs (Table 1) and the lower watershed is almost entirely urbanized.

Table 1
Reservoirs Located in Guadalupe Drainage Basin

Reservoir Name	Basin Area Above reservoir (sq. mi.)	Year Built	Original Capacity (acre-ft)	Current Capacity (acre-ft)
Lexington	37	1952	20,200	19,830
Guadalupe	6	1935	3,700	3,230
Almaden	12	1935	1,800	1,550
Calero	8	1935	10,160	10,050

Combined, the four reservoirs cover a sub-basin area of 63 square miles, almost 40% of the watershed, and have a storage capacity of approximately 35,860 acre-ft. This capacity is used primarily for water supply, with some flood control benefits, although none of the reservoirs are operated for flood control purposes in the wet season. The non-regulated area of the watershed in Santa Clara Valley is largely urbanized and the Guadalupe River and its tributaries have been extensively channelized and leveed. Near the community of Alviso land use changes abruptly from densely urbanized municipal and residential areas to salt ponds and tidal marsh.

The Lower Guadalupe River between San Francisco Bay and Interstate 880 can be divided into two distinct sub-reaches -- tidal and non-tidal. The tidal reach extends upstream from the Bay to approximately the Montague Expressway (12,000 meters) and the non-tidal reach extends from Montague to Interstate 880 (5,600 meters) (Figure 3). As the Guadalupe River enters the tidal zone, where its elevation is within the zone of influence of Bay tides, its slope abruptly shallows causing a decrease in sediment transport capacity that results in significant gravel deposition between Trimble Road and Montague Expressway. The tidal reach exhibits a continued reduction in channel slope with distance downstream and is characterized by tranquil low flow conditions with fine to very fine-grained bed and bank material. Typical cross-sections show relatively flat, narrow strips of tidal marsh bordering the low flow channel, particularly downstream of Tasman Drive, whereas upstream sections have more irregular topography. The non-tidal reach exhibits channel invert elevations above tidal influences and a steeper channel slope with higher energy conditions, reflected by the gravel bed material of the low flow channel, small gravel bars, and occasional steep, eroded banks such as those observed upstream of the Airport Parkway bridge.

Vegetation is also affected by tidal conditions. In fresh water areas upstream of Montague Expressway, riparian woodland consisting largely of willow, cottonwood, and box elder is dominant. Fresh water marsh is found from Montague to the Hetch-Hetchy crossing, characterized by abundant bulrushes, areas of cattails along the channel bottom and decreasing riparian tree cover. Levee banks are covered by weeds and grasses and occasional brush. Salt marsh species become increasingly apparent downstream of Hetch-Hetchy, where cord grass and pickleweed compete with freshwater marsh species. Few, if any, trees are present in this area. Downstream of Tasman Drive, salt water species dominate and trees are absent.

2.2 Geology

2.2.1 General

The Santa Clara Valley is located in a structural depression of the Coast Range that has slowly infilled with sediments, creating a broad alluvial valley floor with deposits several tens of meters thick (Norris and Webb, 1990). A detailed discussion of deposits and sedimentary characteristics in the Santa Clara Valley is provided by Helley and Lajoie (1979).

Rock types vary within the Guadalupe River basin. In the upper basin they consist primarily of sedimentary rocks of the Franciscan Complex. They are highly sheared and faulted, typical of the Coast Range mountains. In the lower valley floor the materials are composed of Quaternary alluvium, some of it partially consolidated (Figure 4). In South San Francisco Bay and Guadalupe River estuary the materials are fine-grained bay muds.

The San Andreas Fault borders the western edge of the basin and the Hayward and Calaveras Faults lie to the east. Numerous other unnamed faults also cross through the basin (DMG, 1992), all of them in a SE - NW orientation that follows the structural alignment of the Coast Range.

2.2.2 Land Subsidence

Santa Clara Valley has experienced regional land subsidence since at least 1906 (Figures 5 and 6). Much of this subsidence is attributed to large-scale groundwater withdrawals that occurred prior to 1965, the year that state water deliveries began to arrive in the San Jose area. Subsidence and ground water table measurements from 1906 to 1995 reveal this trend (Figure 5). Figure 5 shows that almost all of the 14 feet of subsidence measured in downtown San Jose occurred from 1906 to 1965. Since then, the ground water table has reversed its downward trend and about 1 foot of additional subsidence has occurred.

Figure 6 shows that land subsidence along the Guadalupe River has not been uniform, at least for the period 1934 to 1967 (Poland and Ireland, 1968). Subsidence was at a maximum in San Jose at 8 feet (from 1934 to 1967) and lessened with distance outward, approaching 3 feet at the mouth of the Guadalupe River. Land subsidence has altered the bed slope of the Guadalupe River throughout the affected area. Subsidence of the downstream-most portions of the study area has contributed to recent dramatic land encroachment of tidal processes extending to about Montague Expressway.

2.2.3 Sea Level Rise

Throughout the last 10,000 years, the Guadalupe River has adjusted to consistently rising sea levels (Figure 7). Figure 7 shows that mean sea level has risen more than 40 meters during this period. Much of this rise occurred between 7,000 and 10,000 years ago, coinciding with glacial retreat of the last ice age (Atwater et al., 1977). For the past 6000 years, the trend line in Figure 5.3 shows a steady, gradual rise in sea level of 0.1 to 0.2 cm per year, indicating that it will continue to rise into the future. This suggests that sea level will be about 10 to 20 cm (0.4 to .8 ft) higher in 100 years, at the end of the design life of the lower Guadalupe River project. Increases in sea level will result in more extensive landward encroachment of tidal processes (water surface elevation, bay mud deposits and salinity) in the downstream end of the project reach.

2.3 Geomorphology

The lower Guadalupe River exists today as a managed floodway. The natural river channel has been straightened and confined by levees, giving it a relatively simple and consistent cross-sectional geometry that is narrower and deeper than the original stream condition. Historical changes to the channel planform, longitudinal profile, and cross section are described below.

2.3.1 Changes in Channel Planform

Five historical planform channel locations are compared in Figure 8. The channel alignments were obtained from maps prepared between 1857 and 1996 and identified in Table 2.

Figure 8 reveals that the natural river planform has been modified significantly to create a managed floodway. Prior to levee construction in 1963, the river channel was more sinuous and exhibited several meander bends, particularly downstream of Tasman Drive. These meanders appear to have been fairly stable prior to 1963 that may be due in part to the composition of the channel banks. Field observations suggest that the bank materials are a fine-grained silt and clay that may have some erosion resistant properties. Private levees and river training may also have contributed to channel stability in this area during this period.

Table 2
Mapping Examined for Lower Guadalupe River Study

Year Published	Year Planform Data Collected	Title	Source	Scale	Contour Interval	Comments
1870-1875	1857 – 1865	Rancho Ulistac. Rincon de los Esteros	U.S. Surveyor General	variable	N/A	Planform only, no contours
1953	1948	Milpitas. San Jose West	U.S.G.S. Quad Sheets	1:24,000	5 feet	no in-channel topography below water surface
N/A	pre-1963	Lower Guadalupe River Project. Alviso to I- 880	Santa Clara Valley Water District	1"=300'	N/A	Planform only, no contours
1980	1979	Milpitas. San Jose West	U.S.G.S. Quad Sheets	1:24,000	5 feet	no in-channel topography below water surface
N/A	1983 – 1996	Lower Guadalupe River Project. Alviso to I-880	Santa Clara Valley Water District	1"=300'	N/A	Planform only, no contours

In 1963 the levees were constructed along most of the project reach and portions of the river were straightened. Levee construction eliminated the opportunity of high flows to spread onto adjacent floodplain, thereby increasing the effective unit discharge carried by the river for various frequency events. In 1983, sections of levee were realigned and/or raised 0.3 m to 2.4 m, depending on location. In 1995, levees were raised again from the UPRR bridge to Highway 101 by 0.3 to 0.9 m as an emergency measure to provide added conveyance and freeboard. Both the 1979 and 1983-1996 channel alignments in Figure 8 show that the leveed flood control channel generally follows the trend of the historic alignment of the Guadalupe channel, though many of the smaller bends were straightened. Overall, levee construction both straightened and steepened the reach, reducing reach length from the UPRR to Interstate 880 by approximately 25%.

Downstream of Alviso at the north end of the project reach, channel patterns have changed dramatically from their original alignment (Figure 9). At the turn of the century, the tidal zone downstream of Alviso was characterized by a complex distributary channel flow pattern, one in which the number of flow channels increase with distance downstream. Presently, levees confine the flow to a single channel, Alviso Slough, flowing from Alviso to San Francisco Bay.

2.3.2 Changes in Channel Invert

Since significant levee modifications were made to increase flow capacity in 1983, the lower Guadalupe River has shown a general decline in elevation of the invert profile, most notably downstream of Trimble Road (Figure 10). Although some of this decline can be attributed to land subsidence (1 foot or less) during this period, a comparison of channel cross-sections from 1983, 1987 and 1996 shows that the majority is the result of deepening of the low flow channel (NHC 2000).

Because of significant land subsidence in the 1960's and prior, it is not possible to reliably correlate invert elevations from the original 1963 construction plans with 1983 and 1996 data. However, the influence of historic land subsidence on the river channel can be assessed by plotting a profile of land subsidence from Figure 5 along the channel invert (Figure 11). Figure 11 shows that historic land subsidence along the study reach varied from 0 to 8 feet from 1934 to

1967. Given that this period is roughly half that of total land subsidence (see Figure 5), the total is probably closer to double this value. Because the center of maximum subsidence is located upstream of Interstate 880, the river downstream has experienced an overall reduction in channel slope whereas the reach upstream of Highway 880 has experienced an increase. In general, river reaches where slope is reduced tend to exhibit a trend toward decreased stream energy with less bed and bank erosion and greater potential for stream aggradation, whereas river reaches that are steepened tend to show an opposite trend. Identification of these trends on the Guadalupe River cannot be made however, because of artificial channel changes made during levee construction in 1963, immediately following the period of historical land subsidence.

2.3.3 Changes in Channel Cross-Section

Between 1983 and 1996 there is evidence that the channel has increased in size upstream from Trimble Road due to erosion and has decreased in size downstream due to sediment deposition. For the previous sediment investigation by NHC (2000), cross sections surveyed in 1983/87 were compared with cross sections measured in 1996. The 1983 sections are between Gold Street and Airport Parkway, the 1987 sections between Airport Parkway to Interstate 880, and the 1996 cover the entire reach. The 1983 and 1987 cross-section data were obtained from the U.S. Army Corps of Engineers (USCOE, 1989) and 1996 data were obtained from the Water District. All data were converted to NAVD88, where applicable, using the relation Elev$_{NAVD88}$-Elev$_{NGVD29}$=0.81m. In general, the 1996 cross-sections exhibit a more well-defined low flow channel as well as aggradation along the channel banks when compared to 1983/87. This is particularly well illustrated in the tidally influenced zone downstream of Tasman Drive (Figure 12). In addition, almost all of the lateral bench (aggraded bank) deposits located downstream of Tasman Drive exhibit a nearly constant surface elevation from 2.25 and 2.75 m. This consistency of the lateral bench elevation is clearly related to tidal processes and defines an equilibrium elevation for sediment accumulation and the development of marginal marsh features within the leveed channel system.

The difference in cross-sectional area from 1983/87 to 1996 for all stations was calculated to identify sites of overall erosion or deposition. Area differences were extrapolated between cross-sections using the end-area method to calculate volumetric changes along the river. The results are shown in Figure 13 where the continuous line shows the cumulative volume difference (1983 minus 1996) with distance downstream from Interstate 880. This line is negative for areas of net erosion and positive for areas of net deposition. The dashed line shows the net deposition per channel length. Thus, where this line trends down it indicates net erosion whereas an uptrend indicates net deposition. Examination of the cumulative volume difference trend line in Figure 12 shows overall erosion from 1987 to 1996 occurring upstream of Trimble Road. The volume difference trend line rises above zero at Trimble Road and continues to climb with distance downstream, indicating net deposition from Trimble Road to Gold Street. While the volume difference line indicates general trends, the deposition per channel length line reveals local variability in deposition and erosion for shorter reach lengths.

2.4 Current and Future Trends

2.4.1 Land subsidence, Sea Level Rise, and Seismicity

The processes of land subsidence, sea level rise, and deepening of the channel invert all contribute to an overall lowering of the Guadalupe River relative to sea level. This will result in upstream migration of the tidal zone as well as vertical accretion of the tidal marsh over time. Although land subsidence has reduced dramatically since the early 1960's, it has continued in San Jose at a rate of .05 feet per year since 1975 (see Figure 5). Assuming this rate remains constant for the next 100 years, the design life of the project, then 5 feet of additional land subsidence could be expected to occur. In addition, assuming sea level will continue to rise at an approximate rate of .1 to .2 cm per year (see Figure 7), then the 100-year total sea level increase will be on the order of 0.4 to 0.8 feet. Combined, land subsidence and sea level rise yield a cumulative drop in channel elevation relative to sea level of 5 to 6 feet (1.5 to 1.8 m) at San Jose in 100 years. Given that land subsidence has not been uniform historically (see Figure 6), this decline is likely to be less with increasing distance downstream from San Jose. Thus, given the likelihood for continued subsidence and sea level rise, general aggradation and a reduction in channel capacity of tidally influenced areas of the Guadalupe River is anticipated. In addition, vegetation changes can also be expected to occur with upstream migration of the tidal zone.

The San Francisco Bay region is one of the most seismically active areas in North America. Thus, the potential for earthquakes to cause geologically based channel changes is high. Earthquakes can cause significant lateral and/or vertical coseismic adjustment of the ground surface. Liquefaction effects associated with ground shaking may also be produced in fine-grained tidal deposits or areas of loose, unconsolidated fill. This may cause localized settling of channel levees as well as redistribution of in-channel bed and bank sediments along the Guadalupe River.

2.4.2 Channel Cross-Section and Sedimentation

As shown in Figure 13, the lower Guadalupe River is generally aggrading downstream of Trimble Road. Two significant morphologic features are almost always present in this reach. First, there is the development of a well defined low flow channel. Second, there is aggradation of the channel banks, forming bench deposits that border the low flow channel (see Figure 12). The tendency for the river to adopt this type of cross-section should be considered as part of any channel modification plan for this section of river. Channel alterations that diverge significantly from this morphology will require more upkeep and maintenance than do plans that incorporate the natural tendency of the river system.

Conversely, areas upstream of Trimble Road tend to show general erosion with time (see Figure 13), due largely to a steeper channel profile. These areas also show a consistent trend toward development of a low flow channel, similar to areas downstream, but show greater erosion rather than aggradation of the channel banks. Some forms of biotechnical channel invert stabilization may need to be considered for this reach.

One of the primary objectives of the sedimentation study recently completed by NHC for the Water District was to determine if sediment will accumulate in the project reach and weather this will increase design water surface elevations. To evaluate this, NHC constructed a HEC-6T computer model of the project reach which allows for the computation of future long term sediment trends. The results suggest that without regular sediment maintenance, over the next 100 years the long term accumulation of watershed derived sediments in the reach between Interstate 101 and the Montague Expressway may increase the 100-yr flood stage 5 to 7.5 ft (1.5 to 2.3 m) (see Figure 14). This, assuming the levees are high enough to contain the flow. As noted previously, this reach is a natural location for sediment to accumulate due to the reduction in channel slope and flow velocity. Downstream from Tasman only slight increases in the design water surface elevation are expected as the result of river sediment deposition, on the order of 1 ft or less over the next 100 years (Figure 15). As shown in Figure 16, mean tide levels have been increasing over the past 150 years, and at a somewhat increased rate since 1920. However, even since 1920 the average increase in tide has only been approximately 0.007 ft (2 mm). Therefore, increases in the design water surface elevation due to tidal processes are also expected to be minimal (Figure 16).

3.0 Historical Flooding Problems

The project reach has not experienced any significant out of channel flooding problems since the levees were constructed in the 1960s and subsequently raised in the 1980s and 1990s. Since 1940, there have been ten noteworthy floods (see Table 3), the largest occurring on March 10, 1995 (SCVWD, 1997). The USGS estimated the instantaneous peak discharge of the March 1995 flood to be 11,000 cfs at the Guadalupe River gaging station downstream from Los Gatos Creek. This corresponds to a recurrence interval of about 30 years according to flood frequency estimates published by the U.S. Army Corps of Engineers (USCOE, 1977) (see Part 2 Section 3 of this report).

Table 3
Guadalupe River – Notable Peak Flows during past 60 Years

Date	Discharge United States Geological Survey Gage Downstream of Los Gatos Creek, m^3/s (cfs)	Estimated Frequency* (years)
February 27, 1940	246 (8,680)	18
February 2, 1945	187 (6,600)	10
January 12, 1952	227 (8,000)	14
April 2, 1958	259 (9,150)	20
February 19, 1980	224 (7,910)	13
March 31, 1982	208 (7,340)	12
January 24, 1983	202 (7,130)	12
February 18, 1986	259 (9,140)	20
January 9, 1995	263 (9,290)	19
March 10, 1995	312 (11,000)	29

* Based upon Corps of Engineers published hydrology

4.0 Summary Part 1

In summary, the following were important points of the history and future trends of the basin and river:

- The Guadalupe River basin has undergone tremendous land use changes since the turn of the century, including dam and reservoir construction in the upper watershed, urbanization of Santa Clara Valley, and river training and levee construction along all major streams.

- Geology of the upper Guadalupe River basin is characterized largely by sedimentary rocks of the Franciscan Formation, volcanic and ultramafic rocks, sandstone, and alluvium. These rocks are deeply faulted and fractured and bordered to the west by the San Andreas Fault and to the east by the Hayward and Calaveras Faults.

- Historically active fault systems have a high likelihood of causing coseismic adjustment to the Guadalupe River basin within the design life of the project. Historically, coseismic adjustment is thought to have contributed to channel plan and profile adjustments elsewhere along the Guadalupe River.

- The lower Guadalupe River is affected by sea level rise which continues to occur at an estimated rate of 0.1 to 0.2 cm per year. Similarly, the Santa Clara Valley is affected by differential land subsidence, occurring at a rate of 0.05 feet per year in San Jose. If both sea level rise and land subsidence continue, then upstream migration of the tidal zone is expected to cause tidally induced aggradation.

- Levee construction in 1963 reduced channel length by 25%, mainly in the area downstream of Tasman Drive where several historic meanders were present. Levees also eliminated the former distributary flow pattern of the Guadalupe River downstream of Alviso and the ability of high flows to spread onto adjacent floodplains upstream of Alviso.

- The tidal reach of the project reach (UPRR to Montague Expressway) is a depositional environment, characterized by low slope, low energy conditions with a well-defined low flow channel and aggraded channel banks. Bed and bank material consists primarily of silt and clay.

- The non-tidal reach (Montague Expressway to Interstate 880) exhibits greater channel slope and higher stream energy, as well as both erosional (e.g. Interstate 880 to Airport Parkway) and depositional zones (e.g. Trimble to Montague). Bed and bank material consists largely of sands and gravels.

- A comparison between 1983 and 1996 channel geometries shows general declines in invert elevation of 0 to 2.5 m, primarily downstream of Trimble Road. This is mainly due to localized incision of the channel invert rather than general bed degradation.

- Volumetric comparisons made from 1983/87 and 1996 cross-section data show general erosion to have occurred from Interstate 880 to Trimble Road, whereas deposition occurred from Trimble Road to UPRR. This deposition almost always occurred in the form of aggrading and widening channel banks that encroached the channel. The tendency for the river to adopt this type of cross-section should be considered as part of any channel modification plan for this section of river.

- Without regular sediment maintenance over the next 100 years, accumulation of watershed derived sediments in the reach between Interstate 101 and the Montague Expressway may increase the 100-yr flood stage 5 to 7.5 ft (1.5 to 2.3 m). Downstream from Tasman only slight increases in the design water surface elevation are expected as the result of river sediment deposition and tidal processes.

PART 2 – BASELINE HYDRAULIC EVALUATION

1.0 Approach

A HEC-RAS water surface profile computer model was created for the project reach (see Figure 2) to evaluate the capacity of the existing system and evaluate the potential benefits of flood protection alternatives. Part 2 of this report describes existing flood protection features along the project reach, hydrology of the system, and the development of the HEC-RAS model. Evaluation of flood protection alternatives is not addressed in this baseline existing conditions report, as this will be done in a future document.

Two HEC-RAS models were created for the project reach. The first model represents the physical characteristics of the project reach that were present in 1996. This is the most recent date for which channel geometry data are available and is just a year after the flood of record occurred that was used for model calibration. The second "Baseline" model was also developed. It is identical to the 1996 model but includes the new Skyport Bridge (this bridge did not exist in 1996) and a future vegetation planting plan commonly referred to as the Corps' Reach A SRA mitigation planting. The SRA (shaded riverine aquatic) habitat has not been installed but has been approved and should be planted soon. The Baseline model represents the current channel condition and was used to estimate the existing capacity of the project reach with no further channel or levee improvements.

The HEC-RAS model cross-sections are oriented facing in a downstream direction. As such, all references in this report to the "left" or "right" (levees, overbank, etc) are also with respect to a downstream-facing orientation.

2.0 Flood Protection Features

Presently, flood protection is provided along both banks of the river by continuous parallel earthen levees that extend along the entire project reach. However, for the current channel condition, bridge constrictions and low levees contribute to possible breakout flows during the design discharge. The levees were originally built in 1963, but sections were raised in the 1980s and 1990s. No other major flood protection features are present along the project reach.

3.0 Hydrology

Hydrology published by the U.S. Army Corps of Engineers (USCOE, 1977) includes storm frequencies and associated peak discharge rates shown in Table 4 in both metric (System International) units and English units.

Table 4
Lower Guadalupe River Flow-Frequency Relationship

Event Frequency (USCOE, 1977)	**Instantaneous Peak Discharge** $(m^3/s)^{(A)}$	$(cfs)^{(B)}$	**Comment(s)**
2-year	65.14	2,300	
5-year	127.44	4,500	
10-year	189.74	6,700	
25-year	283.20	10,000	
Approx. 30-year	311.52	11,000	10 March 1995 Storm Event
50-year	382.32	13,500	
100-year	481.39	17,000	

(A) – units are cubic meters-per-second (B) – units are cubic feet-per-second

These values represent discharges on the Guadalupe River downstream of Los Gatos Creek (near downtown San Jose) and are prescribed by the Water District at Interstate 880. They are assumed to include local drainage inflows from city/county pump stations discharging into the Guadalupe River at various points. Figure 17 depicts the Corps' flow-frequency relationship, and also includes peak data collected at the downtown San Jose streamflow gaging station (USGS gage 11169000) during the period 1930 to 1995. Figure 18 shows Guadalupe River annual instantaneous peak discharges at the San Jose gaging station for 1930 through 1997. The mean annual instantaneous peak discharge for this period is approximately 3,479 cfs (98.53 m^3/s). It is worth noting that this mean *annual* peak discharge from the USGS gaging record is 1.5 times greater than the Corps' (published in 1977) *2-year* peak discharge of 2,300 cfs. Furthermore, the March 1995 record flood discharge of 11,000 cfs (311.52 m^3/s) plots as a 70-year recurrence interval according to updated frequency analysis of the USGS data (through 1995), whereas a discharge of that same magnitude corresponds to only a 30-year recurrence interval when interpolated from the Corps' published hydrology (Figure 17).

The Water District has specified, in Directive #1 (see Appendix A), the design discharges to be used for the present study. Within the upper part of the study, Reach A, the design discharge corresponds to the Corps' published 100-year (one percent) flood of 17,000 cfs (481.39 m^3/s). This is the same discharge used for the design of other reaches of the Guadalupe River, including the Corps' recent bypass project through downtown San Jose. According to Directive #1, additional flows are to be added incrementally downstream of Highway 101 up to a flow of 20,000 cfs (566.34 m^3/s) below Tasman Road. This is to account for storm water pump station inflows from CalTrans and the Cities of Santa Clara and San Jose. Table 5 gives the SCVWD-specified discharges that form the basis of the analysis herein.

Table 5
Lower Guadalupe River Flood Protection Project Design Flows
(specified by SCVWD)

Location	Design Discharge $(m^3/s)^{(A)}$	$(cfs)^{(B)}$
Downstream of Los Gatos Creek (for present study, below I-880 stn. 17,612)	481.39	17,000
Downstream of Highway 101 (station 14,501)	509.70	18,000
Downstream of Montague Expwy (station 11,807)	538.02	19,000
Downstream of Tasman Road (station 9,222)	566.34	20,000

(A) – units are cubic meters-per-second (B) – units are cubic feet-per-second

4.0 HEC-RAS Model Development

4.1 Cross-Section and Bridge Geometry

The HEC-RAS model used in this study is a modification of the original Water District HEC-2 backwater model dated 1997. Cross-sections in the model are generally spaced 30 meters apart throughout the study reach. The cross-sections were developed from January 1996 photogrammetric mapping data (generally every 30 meters), and merged with May 1996 channel survey data of the low flow channel. The low flow channel was apparently surveyed for every third model cross-section (i.e. surveyed every 90 meters) throughout most of the study reach, with the low flow channel for the remaining two cross-sections between each survey determined by interpolation methods.

The Water District HEC-2 model consisted of five separate input files, each covering a sub-reach of the Lower Guadalupe River. For the previous sediment engineering study recently completed by NHC for the Water District (NHC, 2000), the five input files were merged to form one continuous HEC-2 input deck, extending from San Francisco Bay upstream to Interstate 880. Modifications were also made to the bridge data in the HEC-2 input deck, and cross-sections at the UPRR (formerly SPRR) and the Gold Street bridges were updated with more recent surveys. The bridges were modified from predominantly special bridge method with one representative composite pier to a normal bridge method with individual piers specified. Refinements were also made to contraction and expansion coefficients. Furthermore, many cross-sections were "cleaned up" by altogether removing ineffective cross-section points located outside of the levees.

For the present study, this modified HEC-2 input file was imported into HEC-RAS, which is the Corps of Engineers' successor to the HEC-2 modeling software. Once in HEC-RAS, further refinements were made to the model. Each bridge was compared to as-built bridge plans, if available. Where necessary, refinements were made to the bridge decks and piers to more accurately define the bridge opening. Bridge pier widths were initially assigned to depict the same amount of debris as was assumed by the Water District in their original HEC-2 models.

4.2 Levees

Earthen flood control levees border both banks of the channel along the entire project reach. In order to compare the height of the levees to design water surface elevations, longitudinal profiles were created for both levees. This was accomplished using the "levee" option in the HEC-RAS model, an option in which the user identifies the points on each cross section that represent the top of each levee. As noted in Section 4.1, the cross sections were compiled by the Water District from 1996 topographic mapping. Therefore, the accuracy of the levee profiles is limited to the accuracy of the available mapping. The HEC-RAS levee profiles appear reasonable when compared with profiles of the levees that have been created from a variety of other data sources. However, much more detailed field surveys are required to establish accurate levee profiles for final design and construction.

During high flows, water will overtop a low section of levee on the west (left) bank a short distance downstream from the UPRR bridge near Alviso. This is the only documented location within or near the study reach of significant levee overtopping during recent flood events. This overtopping location was simulated in the HEC-RAS model by adding a junction and overflow branch at the location of the levee overtopping. The overflow portion of the levee was then inserted to the overflow branch as an inline weir spillway. Other portions of levee that may potentially overtop during design event flow conditions were not accounted for. In other words, all of the flow was confined to the channel by artificially assuming that the levees had infinite height (often referred to at the "glass-walled" scenario).

4.3 Calibration

The HEC-RAS model was calibrated to water surface profile data from the March 10, 1995 flood of record. High water mark data from the event were provided by the Water District and extend from station 6,600 near Alviso to 17,612 at Interstate 880. The Water District estimated the high water mark elevations using photogrammetry and aerial interpretation, and they correspond to peak water levels along the left bank, center, and right bank portions of each cross-section. These three elevations were averaged at each section (throughout most of the reach, there is no significant lateral variability in the data per cross-section) and input as observed data to the HEC-RAS model. The peak discharge during that event was estimated by the USGS at their Guadalupe River at San Jose gage (below Los Gatos Creek) to be 11,000 cfs (311.5 m^3/s). This was the flow rate used along the entire project reach and no adjustments were made for local inflows from storm water pump stations, because the inflows were not known and are not believed to have significantly affected the peak. To estimate the tailwater elevation during the flood downstream from the project reach, an estimate had to be made of the amount of water that overtopped the west levee downstream of the UPRR bridge. Trial and error simulations were required with the model to balance the junction (the model includes the breakout as a separate reach with the overtopping levee as a weir). The final calibration run resulted in an estimated overflow of about 3,215 cfs (91 m^3/s), with the remaining flow of 7,785 cfs (220.5 m^3/s) continuing down Alviso Slough to the Bay. It is possible that this overestimates the amount of overflow actually lost, as the overbank (weir) elevations at this location may have been higher in 1995 than at present (possibly combined with a lower tide). This does not affect calibration in the project reach upstream of UPRR. For this event (and the verification events - see next section) the approximate Mean Higher High Water (MHHW), 2.3 m NAVD 88, was assumed for tailwater conditions at the Bay, although tidal influences had no significant impact on water levels within the project reach.

Estimates of channel resistance were initially made based upon field inspections of the channel roughness, aerial photo interpretation, and references literature. At each cross-section, Manning's *n* resistance coefficients were specified separately for the low flow channel and the overbank rather than a single composite *n* value for the entire cross-section as typically prescribed by the Water District. Cross-section resistance on the overbanks was further subdivided within the Gold Street to Montague Expressway reach. Separate *n* values were specified for the minimally vegetated levee side slopes, and the remainder of overbank (between levee toe and low flow channel bank) which is dense with bulrush and other vegetation. This procedure allows a more accurate assessment of any alternatives that involve modifications on either part of the cross-section. Elsewhere in the project reach, the resistance across the entire overbank is found to be more uniform. Furthermore, an attempt was made to keep Manning's *n* values constant on a reach-by-reach basis if the vegetation did not change significantly from cross-section to cross-section.

To calibrate the HEC-RAS model it was run for the March 1995 flow conditions. Initial computed results compared fairly well to the high water data in most areas. Slight refinements were made to the Manning's *n* values in places to better refine the calibration. The final *n* resistance values are presented below in Table 6.

Table 6
Manning's *n* Resistance Coefficients for Calibrated HEC-RAS Model

Sub-Reach	Low-Flow Channel	Overbank
UPRR (stn 7005) to Gold St (stn 7115)	0.03	0.80
Gold St (7115) to Hwy 237 (8010)	0.03	0.20 (0.04 levee)*
Hwy 237 (8010) to Tasman (9240)	0.027	0.20 (0.04 levee)*
Tasman (9240) to Station 9960	0.03	0.25 (0.04 levee)*
Station 9990 to Station 10800	0.03	0.20 to 0.10 (0.04 levee)*
Station 10830 to Station 11670	0.027 to 0.03	0.06 (0.04 levee)*
Station 11700 to Montague (11825)	0.03	0.08 (0.04 levee)*
Montague (11825) to Trimble (13700)	0.05 to 0.06	0.03 to 0.06
Trimble (13700) to Hwy 101 (14525)	0.03	0.05
Hwy 101 (14525) to Parking Lot (14695)	0.02	0.03
Parking Lot (14695) to Interstate 880 (17630)	0.027	0.027

* At these locations, a value of 0.04 was assigned to the side slope face of the levees. The remaining value(s) stated represent the rest of the overbank (i.e., bench, etc.).

During the course of calibration, it was necessary to reduce at two of the bridges the amount of pier debris assumed. Debris at westbound Highway 237 was reduced from approximately 2 m to 1 m per pier, while Tasman was reduced to 0.5 m. No direct measurements of debris width were taken for the 1995 flood; Table 7 presents the debris amounts per pier assumed at each bridge. The computed water surface profile for the calibrated March 1995 condition, compared to the high water mark data, is presented in Figure 19.

Table 7
Debris Amounts per Pier Assumed at Each Bridge

Bridge	Debris Width Per Pier
UPRR	2 m
Gold	1 m
Hwy 237 (west bound)	1 m
Hwy 237 (east bound)	2 m
Tasman	0.5 m
Montague	1 m
Trimble	1 m
Hwy 101	1 m
Parking Lot	1 m
Airport Parkway	1 m
Skyport	0.5 m
Interstate 880	0.75 m

4.4 Verification

Additional field-surveyed high water mark data for the January 9, 1995 and January 26, 1997 flows were provided by the Water District. The January 1995 data consist of only 18 points along limited sections of the study reach. However, the event is suitable for verifying (or validating) the calibrated model, as it had a substantial peak flow estimated at the USGS gage to be 9,290 cfs (263.1 m^3/s) which is not much lower than the March 1995 calibration event. Using the calibrated HEC-RAS model, a water surface profile was also produced for the January 1995 flow. As shown in Figure 20, the results compare favorably to the high water marks, validating the calibration. Also shown on Figure 20 is a computed profile for a January 1997 event of 5,470 cfs (154.9 m^3/s) and the measured high water marks. These results do not compare as well as the January 1995 verification event however. As illustrated, the computed water surface throughout most of the reach is lower than the high water marks, generally by about ½ meter but by as much as 1 meter in places. This is expected however. The January 1997 peak discharge was only about one-half of the March 1995 calibration event. At these lower discharges, channel roughness is higher because the influence of the vegetation is considerably greater. Therefore, the calibrated HEC-RAS model will provide reasonable water surface elevations for large floods, but will under predict channel roughness and thus water levels for the smaller events. NHC strongly recommends high flow monitoring of flows and water surface profiles, to provide the data necessary to quantify how Manning *n* varies with flow and depth on a reach-by-reach basis.

5.0 HEC-RAS Model Application to Determine Channel Capacity

5.1 Baseline Conditions

The calibrated model reflects channel conditions during the March 1995 flood, which is essentially the same as present time except for the recently constructed Skyport Drive bridge in Reach A and some slight changes in vegetation density and sediment accumulation. The baseline conditions for the Lower Guadalupe River study, from which alternatives will be developed and compared, are defined by the Water District in Directive 3 (see Appendix A). It states that the baseline is to represent the present channel condition plus the proposed SRA (Shaded Riverine Aquatic) vegetation planned within Reach A. The SRA planting is mitigation for the proposed U.S. Army Corps of Engineers' flood control project for the downtown reach of the Guadalupe River (just upstream of Interstate 880). All SRA plantings are proposed to occur within the reach between Airport Parkway and Interstate 880. The calibrated model was modified accordingly, with the resulting baseline model thus differing from the original calibrated model only upstream of Airport Parkway. At this point in the study, debris assumptions remain the same for the baseline model as in the calibration model.

In order to more accurately evaluate potential hydraulic impacts upstream of Interstate 880 due to the SRA mitigation or other components of the project, the baseline HEC-RAS model was extended upstream to Coleman Avenue. The Interstate 880 bridge was added to the model based upon CalTrans bridge plans and information contained within the Watershed Planning Study Engineers Report (SCVWD, 1997). Cross-sections from Interstate 880 to Coleman were taken directly from the existing Corps' HEC-RAS model (developed by NHC) of the downtown San Jose flood control project, with the cross-sections converted to metric units and to the NAVD 1988 datum. Otherwise, this reach of the baseline model (Interstate 880 to Coleman) is exactly the same as the downtown model, with no further refinements.

5.2 SRA Roughness Determination

Simulation of the effects of the SRA plantings required an estimation of the change in roughness, in terms of a Manning's *n* resistance coefficient, for the proposed vegetation. The SRA consists of rows of trees planted along the banks of the low flow channel, providing shade which will help control temperature in the river. Along with trees and other riparian plantings, it is expected that there will eventually be an increase in the undergrowth (brush and shrubs) in the vicinity of the plantings as well. The SRA plan is described in the Corps' mitigation and monitoring plan by the Corps and Jones and Stokes for the downtown project, revised October 1999 (USCOE, 1999). The drawings and written descriptions in the plan formed the basis of the *n* value estimation, which includes both Phases 1 and 2 of the vegetation planting. Significant research has been carried out to estimate resistance characteristics of flow through various types of vegetation. However, the difficulty lies in that every site is unique, and furthermore, in this instance we are analyzing a future condition in which most of the plantings have not yet occurred let alone matured.

As an initial estimate of SRA *n* values, a method which is described by the USGS for selecting roughness coefficients due to vegetation on floodplains (USGS, 1989) and based upon methods developed by Petryk and Bosmajian in 1975 was used. This method adjusts a base n value (without vegetation) to account for the diameter and density of the trees or other vegetation, depth of flow, and drag coefficient of flow around the vegetation. Assuming 1-ft diameter trees (fairly mature), with 50% mortality of the original plantings, a flow depth corresponding to design flow conditions, and average undergrowth beneath the tree canopy, a Manning's n value of 0.14 was estimated. This value corresponds only to the area where the riparian plantings occurred (generally along the low flow banks). Assuming a base n value of 0.035, composite n values representing the entire overbank (but not including the low flow channel) were then determined to be 0.075 for the reaches with 4 rows of plantings, and 0.042 for the upper reach with single row plantings. Based upon further research into other literature combined with engineering judgement, an additional higher (more conservative) estimate was made for the overbank Manning's n values due to SRA: 0.10 along the 4 row reaches, and 0.05 along the single row reaches. Two baseline RAS models were subsequently developed, one for each assumed SRA roughness, which more or less provides a sensitivity analysis under a range of potential SRA conditions.

The calibrated baseline condition HEC-RAS model was then applied for the design flow conditions, ranging from 17,000 cfs (481.4 m^3/s) at the upstream end of the reach to 20,000 cfs (566.3 m^3/s) downstream of Tasman (see Part 2 Section 3 of this report). The final baseline design run resulted in an estimated overflow along the west bank near Alviso of 9,535 cfs (270 m^3/s), with the remaining flow of 10,465 cfs (296.3 m^3/s) continuing down Alviso Slough to the Bay. Tailwater conditions at the Bay were based upon the 10-year tide of 3.1 m NAVD 88 (USCOE, 1984), although under design flood conditions the tide had minimal effect on computed water levels upstream from Alviso. The baseline RAS design water surface profile from the Bay to Interstate 880 (under glass-walled conditions), compared to the levees, is presented in Figure 21. A close up profile from Highway 101 to Coleman is presented in Figure 22, comparing the design profiles under each of the two assumed SRA conditions to the non-SRA condition. Similarly, a close up of the design baseline profile from Alviso (UPRR bridge) to Highway 101 is also presented in Figure 23. Under the more conservative SRA assumption of $n = 0.10$ (Figure 22), the water surface at the downstream face of Interstate 880 remains under the target elevation of 17.86 m but nearly matches this target at the upstream face. Therefore, we are very close to other present criteria at Interstate 880; however, it is important to try to validate some of the vegetation roughness assumptions (field verification is recommended).

5.3 Channel Capacity

The calibrated baseline HEC-RAS model was used to estimate the present capacity of the Lower Guadalupe channel from UPRR to Interstate 880 (*i.e.* the present level of protection of the levee system). A series of six profiles were simulated with the model, ranging in 3,000 cfs increments from 8,000 cfs (226.5 m^3/s) to 23,000 cfs (651.3 m^3/s). The profile results are plotted in Figure 24, compared to the top of levee profiles. Again, it should be noted that for locations where the levee is overtopped, a glassed-wall scenario is assumed. In such places the actual water surface would still overtop the levee, but not to the level depicted in the profile plot. This approach identifies locations where there are present risks of overtopping.

The capacity of the Alviso Slough reach, downstream of the UPRR bridge, is dependant upon the coincident downstream tide at the Bay and cannot be correlated directly to a specific discharge. For the purpose of channel capacity evaluation, however, downstream boundary conditions assumed a normal depth tailwater condition. Under this condition, the model indicates that at a discharge of about 8,000 cfs (226.5 m^3/s) flow would begin to overtop low spots in the Alviso Slough levee system. Upstream of the UPRR bridge, within the project reach, the effects of assumed tide on channel capacity is minimal.

The channel capacities stated herein do not account for any freeboard, but rather they estimate the ultimate level of protection provided by the present levees prior to overtopping (as shown in Figure 24). Should freeboard be added, the actual capacities would be less. As summarized below in Table 8, from UPRR upstream to Highway 237 incipient overtopping corresponds very closely to the 17,000 cfs (481.4 m^3/s) profile. Above Highway 237 and upstream to Montague Expressway, the overall channel capacity drops to about 14,000 cfs (396.4 m^3/s), although with some sections of this reach capable of passing up to about 15,500 cfs (438.9 m^3/s). Channel capacity gradually increases from Montague upstream to Highway 101. Just above Montague the capacity is about 15,000 cfs (424.8 m^3/s), at Trimble about 20,000 cfs (566.3 m^3/s), and just below Highway 101 it is over 23,000 cfs (651.3 m^3/s). Just above Highway 101 the channel capacity drops to about 18,500 cfs (523.9 m^3/s), then increases with distance upstream to about 23,000 cfs (651.3 m^3/s) at Airport Parkway. Capacity in the remainder of Reach A upstream from there exceeds 23,000 cfs.

Table 8
Approximate Channel Capacity by Sub-Reach*
(Between Levees with No Remaining Freeboard)

Sub-Reach	Discharge (cfs)	Discharge (m^3/s)
UPRR (stn 7005) to Hwy 237 (stn 8010)	17,000	481.4
Hwy 237 (8010) to Montague (11825)	14,000	396.4
Montague (11825) to Trimble (13700)	15,000 to 20,000	424.8 to 566.3
Trimble (13700) to Hwy 101 (14525)	20,000 to 23,000	566.3 to 651.3
Hwy 101 (14525) to Airport Parkway (15685)	18,500 to 23,000	523.9 to 651.3
Airport Parkway (15685) to Interstate 880 (17630)	Over 23,000	Over 651.3

* Values represent approximate ultimate capacities within the present levee system for sub-reaches of the project, assuming no remaining freeboard.

Considering the Lower Guadalupe system as a whole, the ultimate channel capacity is approximately 14,000 cfs (396.4 m^3/s), with the most limiting reach located between Tasman and Montague. Upstream of Trimble bridge and through the remainder of the project reach, the channel appears generally capable of passing the design discharge. Over most of this area, freeboard height may be adequate, although the levees would still require FEMA certification. Therefore, it would appear that any flood control alternatives should concentrate primarily on lowering flood levels downstream of Trimble. Considering levee freeboard requirements, the present condition design profile (Figure 21) would need to be lowered by (or the levees raised by) up to 2 m or more in parts of this reach.

5.4 Bridge Capacity

The calibrated baseline HEC-RAS model was also used to estimate the capacity of the existing bridges in the project reach. Model cross-sections of each bridge were compared to computed water levels from the range of events simulated for the channel capacity analysis (see previous section), in order to determine the discharge rate at which the flow changes from free-surface to pressure at each bridge (*i.e.* the discharge that corresponds to a water level just hitting the soffit or low chord of the bridge deck). As mentioned previously the water surface profiles are shown on Figure 24, while Table 9 below summarizes the results of the bridge capacity analysis. The stated capacities could change if debris conditions differ from those assumed (see Table 7).

Table 9
Approximate Existing Bridge Capacities

Bridge	Discharge* (cfs)	Discharge* (m^3/s)	Bridge Deck Overtop During Design Event?	Pressure Flow During the Design Event?
UPRR	19,000–22,000	538.0 – 623.0	No	No
Gold	16,000–18,000	453.1 – 509.7	No	Yes
Hwy 237 (west bound)	18,500–20,000	523.9 – 566.3	No	Yes (barely, at upstream face)
Hwy 237 (east bound)	8,500–9,500	240.7 – 269.0	Yes	Yes
Tasman	13,500–15,500	382.3 – 438.9	No	Yes
Montague	11,500	325.6	Yes	Yes
Trimble	18,000	509.7	No	Yes
Hwy 101	16,500–17,500	467.2 – 495.5	No	Yes
Parking Lot	21,000–23,000	594.7 – 651.3	No	No
Airport Parkway	20,000–22,500	566.3 – 637.1	No	No
Skyport	23,000 – over 23,000	651.3 – over 651.3	No	No
I-880 (soffit estimated)	Over 23,000	Over 651.3	No	No

* First value given per bridge represents discharge at which water surface begins to hit the bridge soffit (low chord).
Second value given represents discharge at which soffit becomes completely inundated/wet (full pressure flow).
Single discharge values given for bridges with horizontal (or nearly horizontal) soffits.
Analysis assumes all flow confined between existing levees (glass-walled scenario described in Section 4.2).

As indicated in the table, the upstream (east bound) Highway 237 bridge is a significant constriction, capable of passing only about half the design event discharge without going to pressure flow. The Montague Expressway bridge also has a limited capacity. These two bridges are also the only ones that are shown to overtop during the simulated design flood event. Table 9 also indicates which bridges exhibit pressure flow conditions during the design event. Except for UPRR at the lower end, all bridges from Highway 101 downstream exhibit pressure flow while the remaining upstream bridges within Reach A remain free-surface. This can also be seen in the design flow profile plots of Figures 22 and 23.

Analysis of flood control alternatives needs to consider bridges in addition to levee capacities. The present free-surface flow capacity of at least two bridges is less than the limiting channel/levee capacity of 14,000 cfs. To remove these bridges from pressure flow will require lowering the design water surface (or, alternatively, raising the bridges) by more than 2 m.

6.0 Summary Part 2

In summary, the following points are important in the development of the HEC-RAS model and channel capacity evaluation:

- The model is in metric units, with all elevations referenced to the NAVD 1988 datum.
- The model extends from San Francisco Bay to Interstate 880 (the model actually extends upstream from Interstate 880 to Coleman Street. This was done by adding on a portion of an existing HEC-RAS model to the project reach model. However, no attempt was made to calibrate or refine the model upstream from Interstate 880).
- Cross-section data are based upon 1996 topographic mapping and field surveys conducted by the Water District. The sections are spaced approximately 30 m apart. The over bank portion of each cross section was estimated from the topographic maps. The low flow channel was field surveyed at 90 m intervals and merged with the appropriate cross sections. The low flow channel was interpolated at cross sections without field surveys.
- The cross-sections are oriented viewing in a downstream direction, rather than upstream as the Water District will often do. As such, all references herein to left and right banks or levees are, therefore, with respect to facing downstream. This is consistent with the Guadalupe HEC-RAS model developed for the downtown San Jose design study, and is also the "industry standard".
- Except for the Interstate 880 to Coleman reach, all of the bridges are simulated using standard HEC-RAS bridge methodologies.
- Longitudinal profiles of the levees have been estimated and appear reasonable, however, a thorough field survey is needed to establish precise heights prior to final design.
- Manning's *n* values and expansion/contraction coefficients were refined through calibration. Dependability of the calibrated model was verified with another high flow event.
- The calibrated model should only be used for large floods. Manning's *n* values will need to be increased if it is to be used for smaller events. Additional data are suggested to be able to determine how vegetation affects model roughness for flows less than about 8,500 cfs (240.7 m^3/s).

- The Baseline model is identical to the calibrated model expect it includes the Skyport Drive bridge and the proposed SRA planting plan which is mitigation for the U.S. Army Corps of Engineers Guadalupe River downtown flood protection project.

- The design flow water surface profile exceeds the height of the existing levees between the UPRR bridge and Trimble Road (under glass-walled conditions) by about ½ to over 1 m. Upstream from Trimble the profile is lower than the levees: however, the present levees may not provide sufficient capacity to comply with standard FEMA freeboard certification requirements.

- Within the project reach, water will begin to overtop the levees at approximately 14,000 cfs (396.4 m^3/s) between Tasman Road and Montague Expressway.

- Beginning just below Airport Parkway upstream through Interstate 880, the existing levees and bridges can contain a flow significantly larger than the design flow, up to approximately 22,500 cfs (637.1 m^3/s) or greater. This provides potential capacity for vegetation mitigation (SRA, CalTrans, etc.), provided target water levels at Interstate 880 for the downtown project are not compromised.

- During the design flow event, all bridges from Gold Street inclusive through Highway 101, exhibit pressure flow conditions. The upstream (east bound) Highway 237 and Montague Expressway bridges are the most constricted in the project reach; both overtop in the design event.

7.0 Recommendations

Successful development of an accurate flood protection plan will require the additional collection and monitoring of certain critical data. The following items are recommended:

- It is highly recommended that water surface profile (high water marks) and flow data be collected within the project reach during this year's runoff season, if possible, preferably over a range of flow rates: high (over 8,500 cfs if occurs), intermediate (4,000 cfs to 8,000 cfs), and low (2,000 cfs or below). This will provide the data necessary to quantify how Manning's *n* varies with depth, resulting in updated calibration of the model for the full range of expected flows – and based on the present channel condition.

- Prior to final design and construction, detailed surveys will be required to establish accurate levee profiles. It is also recommended that the bridges be re-surveyed.

- In order to develop more confidence in assumed roughness for the SRA vegetation, field monitoring during floods should be undertaken to estimate Manning's *n* values for locations along the river where similar vegetation already exists.

- Consideration should be given to monitoring pumped inflow rates during the winter, to get a feel for the significance of these additional flows during a wide range of flow events on the river.

- Cross-section surveys at select key locations should be taken periodically, to monitor trends in sedimentation (*i.e.* erosion and deposition) and cross-section geometry (*i.e.* bench development, low-flow channel).

REFERENCES CITED IN REPORT

Atwater, B.F., Hedel, C.W., and Heeley, E.J., 1977. "Late Quaternary depositional history, Holocene sea-level changes, and vertical crustal movement, southern San Francisco Bay, California," Geological Survey Professional Paper 1014. 15p.

DMG (California Division of Mines and Geology), 1992. "Preliminary fault activity map of California," DMG Open-File Report 92-03.

Helley, E.J. and Lajoie, K.R., 1979. "Flatland deposits of the San Francisco Bay region, California--their geology and engineering properties, and their importance to comprehensive planning," Geological Survey Professional Paper 943, 88p.

Norris, R.M. and Webb, R.W., 1990. *Geology of California*. 2nd ed., John Wiley & Sons, Inc., New York, 541 p.

NHC, 2000. "Lower Guadalupe River Sedimentation Study." Santa Clara Valley Water District Final Report. NHC is an acronym for Northwest Hydraulic Consultants.

Poland, J.F., and Ireland, R.L., 1968. "Land subsidence from 1934 to 1967, Santa Clara Valley, California," U.S. Geological Survey Open File Map.

SCVWD, 1997. "Guadalupe River Watershed Planning Study Engineer's Report, Project No. 3015" SCVWD is an acronym for Santa Clara Valley Water District.

USCOE, 1977. "Santa Clara County, California, Guadalupe River and Coyote Creek Hydrologic Engineering Office Report." USCOE is an acronym for U.S. Army Corps of Engineers.

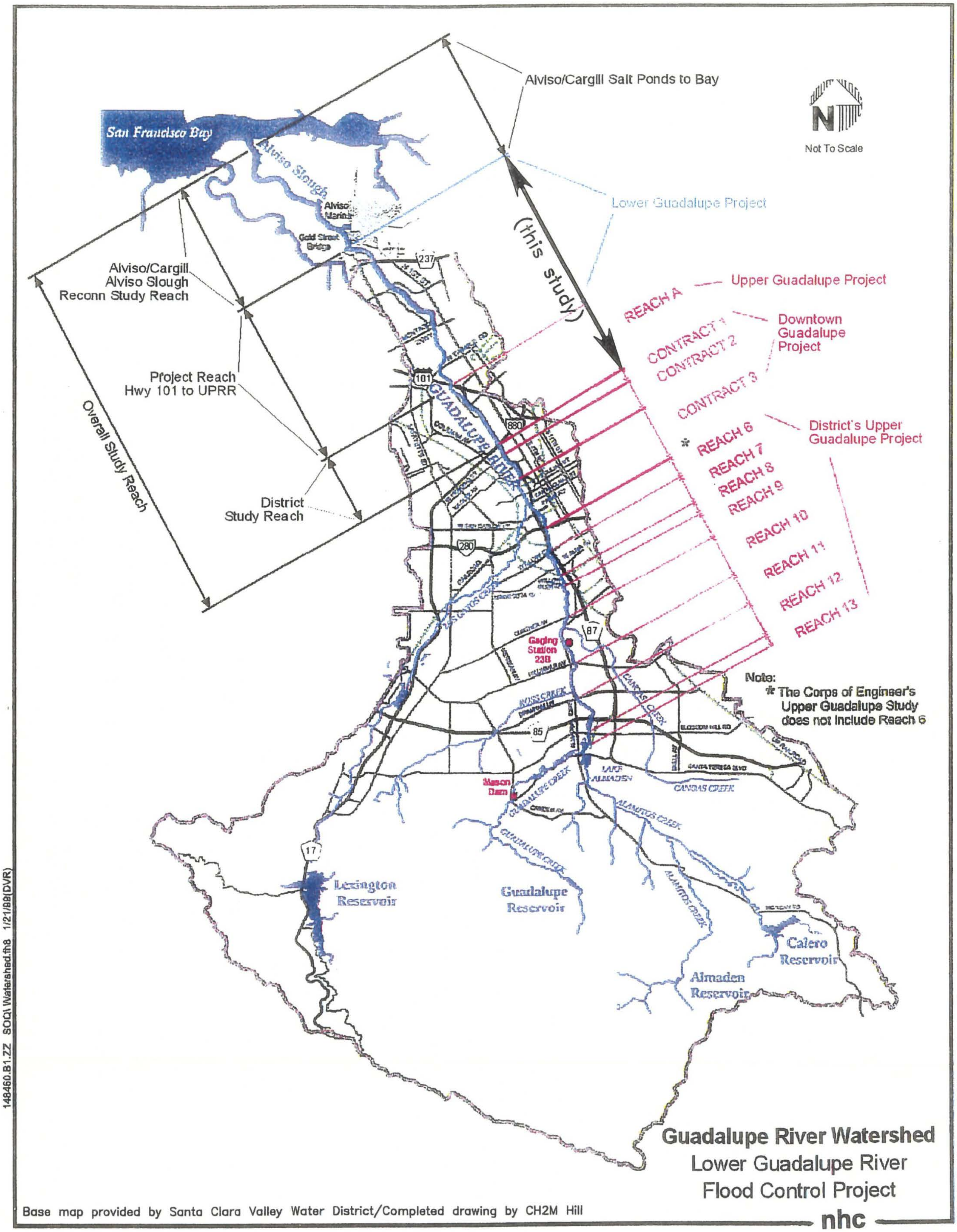
USCOE, 1984. "San Francisco Bay, Tidal Stage vs Frequency Study." USCOE, San Francisco District.

USCOE, 1989. "Hydraulic analyses of the Guad 104 reach along the Guadalupe River," prepared by Water Engineering & Technology, Fort Collins, Colorado, 37 p. plus Appendices.

USCOE, 1999. "Guadalupe River Flood Control Project. Downtown San Jose, California. Mitigation and Monitoring Plan." Prepared by USCOE, Sacramento District, in cooperation with SCVWD.

USGS, 1989. Water-Supply Paper 2339. "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," by George J. Arcement, Jr. and Verne R. Schneider. USGS is an acronym for United States Geological Survey.

FIGURES



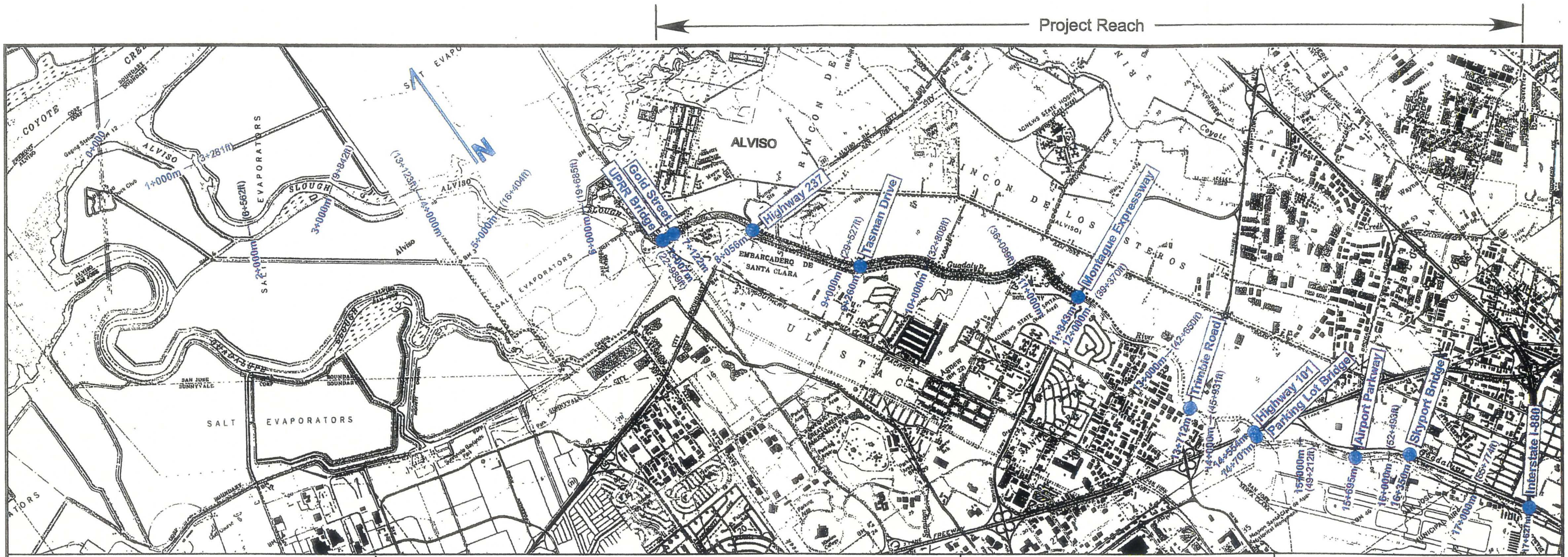
Guadalupe River Watershed
Lower Guadalupe River
Flood Control Project

nhc

Base map provided by Santa Clara Valley Water District/Completed drawing by CH2M Hill

148460.B1.ZZ SOQ1 Watershed.fh8 1/21/98(DVR)

Figure 1



northwest hydraulic consultants

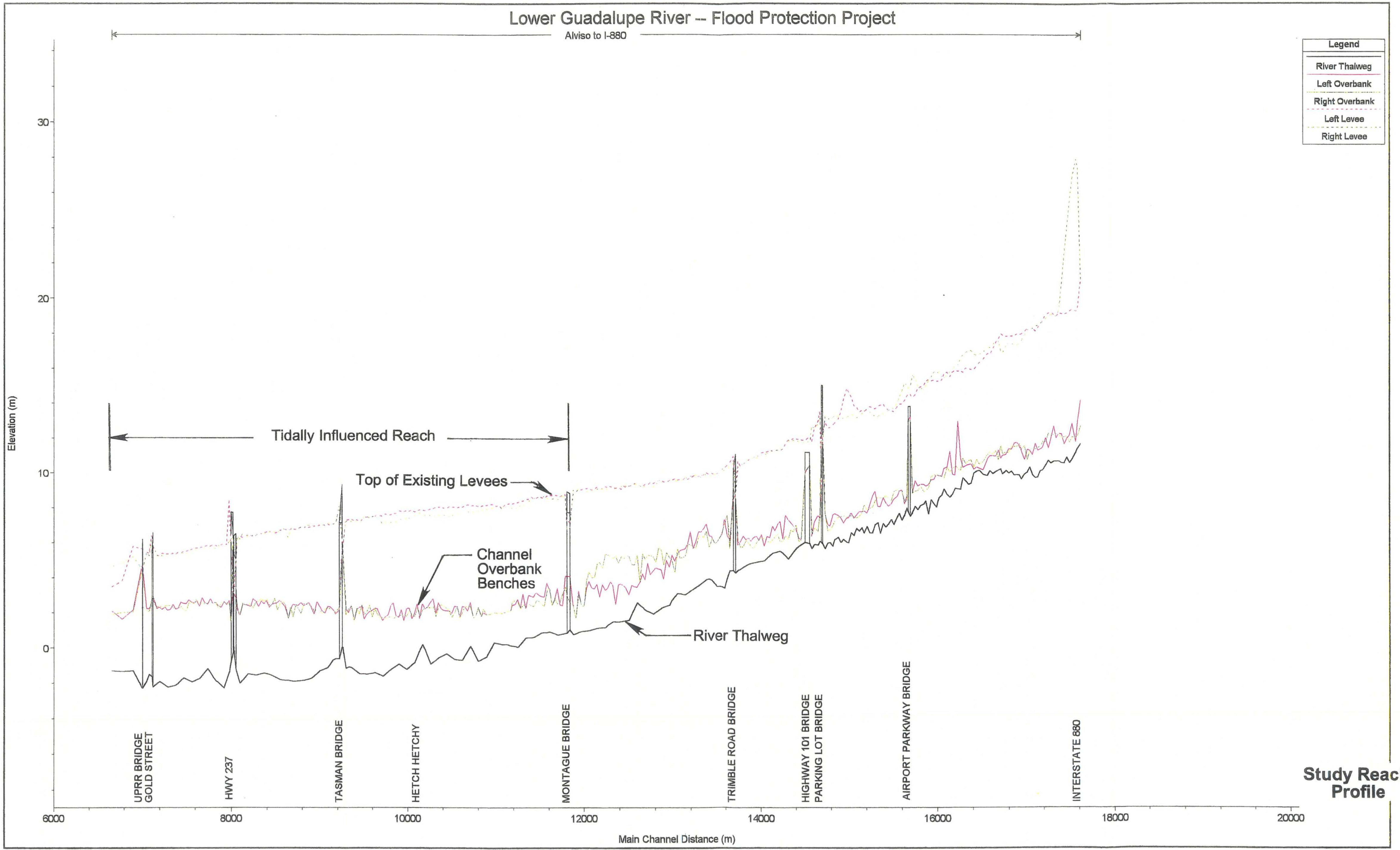
notes:
- base mapping from USGS quadrangle sheets San Jose West, CA; Milpitas, CA; and Mountain View, CA.
- All stationing is in meters, with stationing in feet shown in parentheses using the conversion 1 m = 3.2808
- Stationing indicating bridge locations is approximate.

0 1 2
kilometers

Study area map of Lower Guadalupe River

nhc

Figure 2

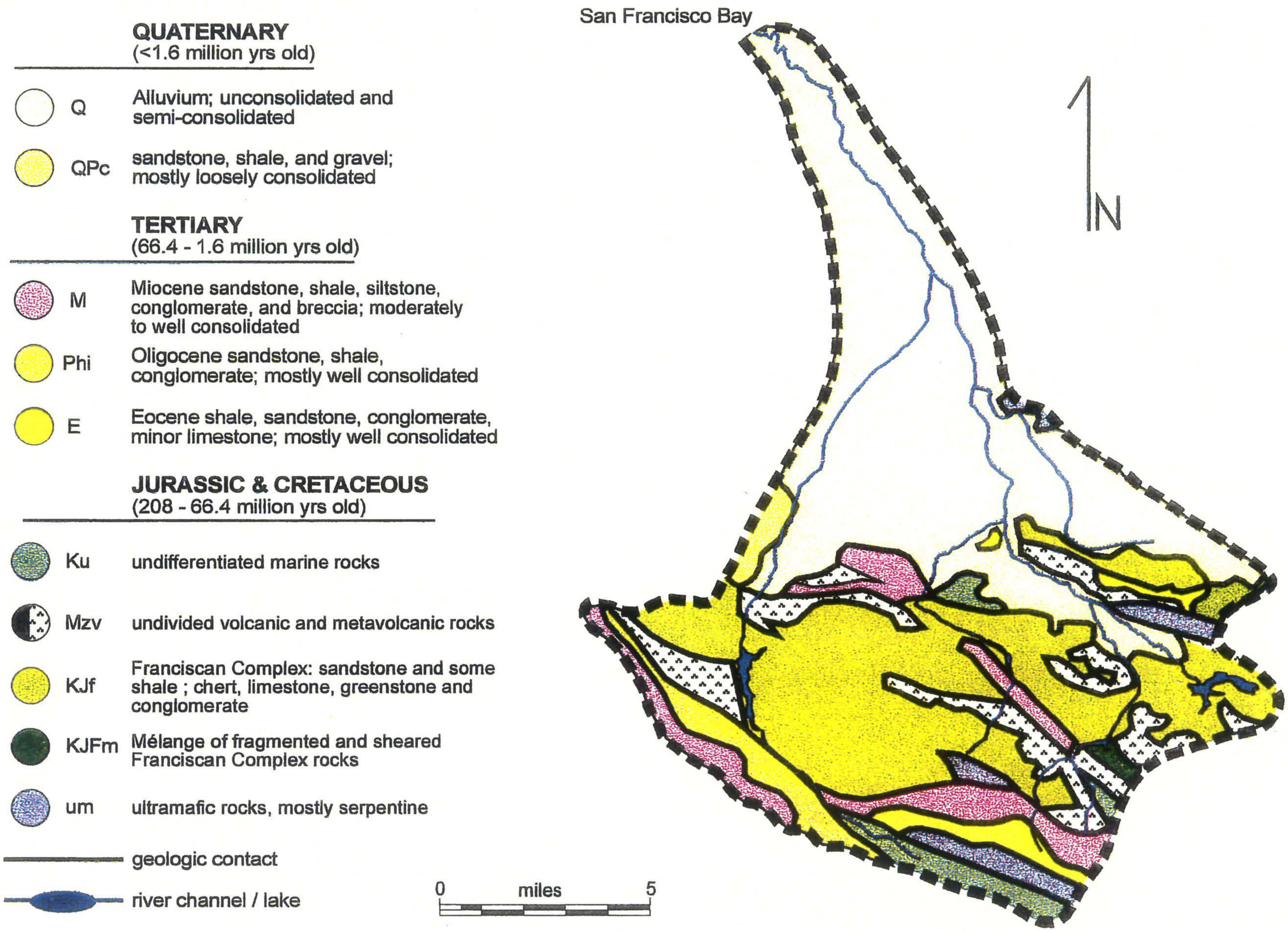


Study Reach Profile

1 cm Horiz. = 400 m 1 cm Vert. = 2 m

Figure 3

Figure 4


QUATERNARY
(<1.6 million yrs old)
Q
Alluvium; unconsolidated and semi-consolidated
QPc
sandstone, shale, and gravel; mostly loosely consolidated
TERTIARY
(66.4 - 1.6 million yrs old)
M
Miocene sandstone, shale, siltstone, conglomerate, and breccia; moderately to well consolidated
Phi
Oligocene sandstone, shale, conglomerate; mostly well consolidated
E
Eocene shale, sandstone, conglomerate, minor limestone; mostly well consolidated
JURASSIC & CRETACEOUS
(208 - 66.4 million yrs old)
Ku
undifferentiated marine rocks
Mzv
undivided volcanic and metavolcanic rocks
KJf
Franciscan Complex: sandstone and some shale ; chert, limestone, greenstone and conglomerate
KJFm
Mélange of fragmented and sheared Franciscan Complex rocks
um
ultramafic rocks, mostly serpentine
geologic contact
river channel / lake
0
miles
5
San Francisco Bay
N

Geologic map of Guadalupe River basin (from DMG, 1990). Faults are not shown.

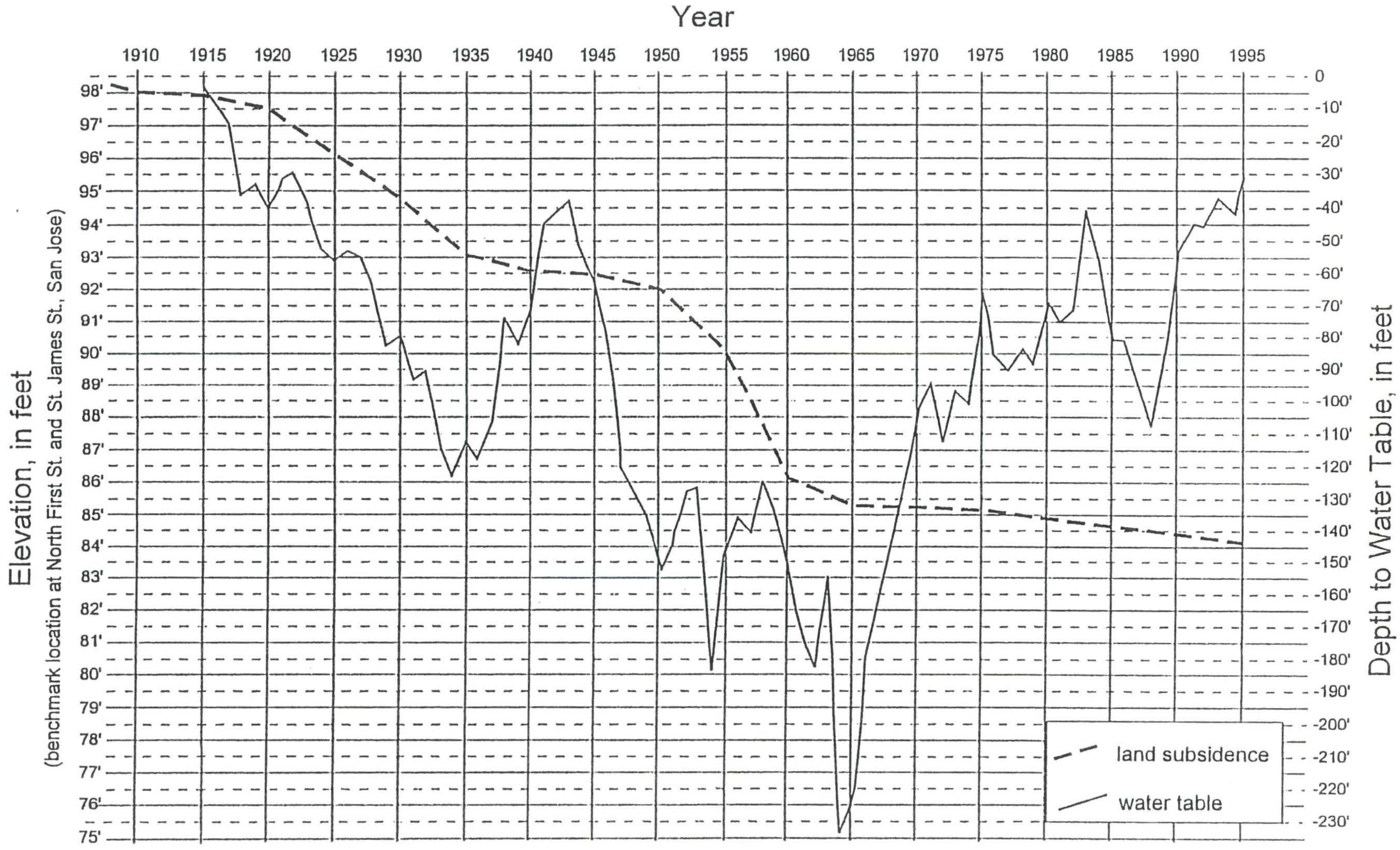
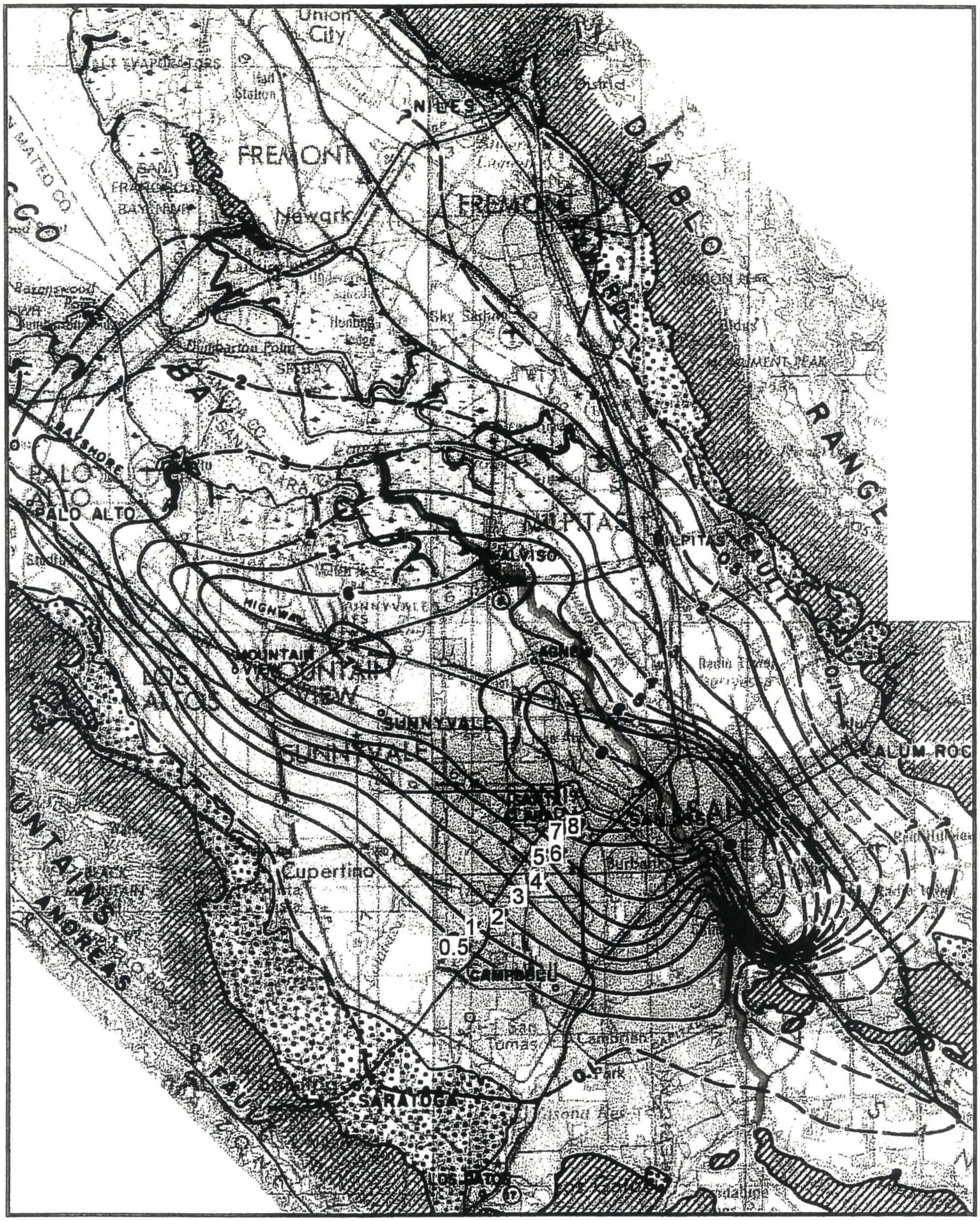


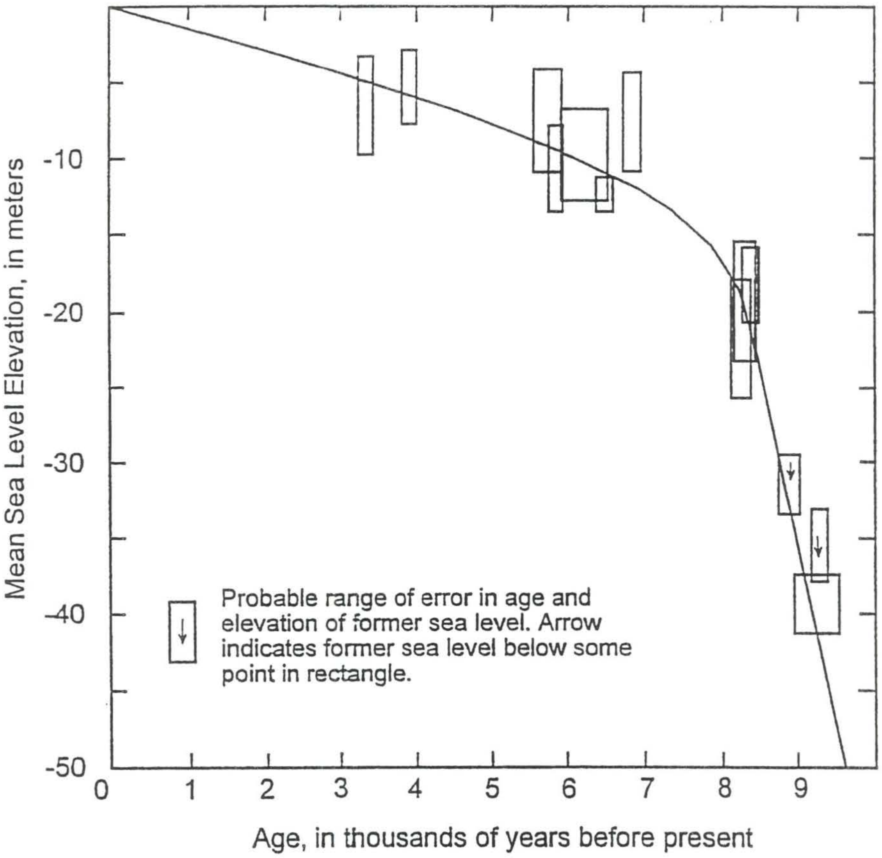
Figure 5

Land Subsidence and Depth to Water Table, San Jose, California (1906 - 1995) (from: SCVWD, 1997).



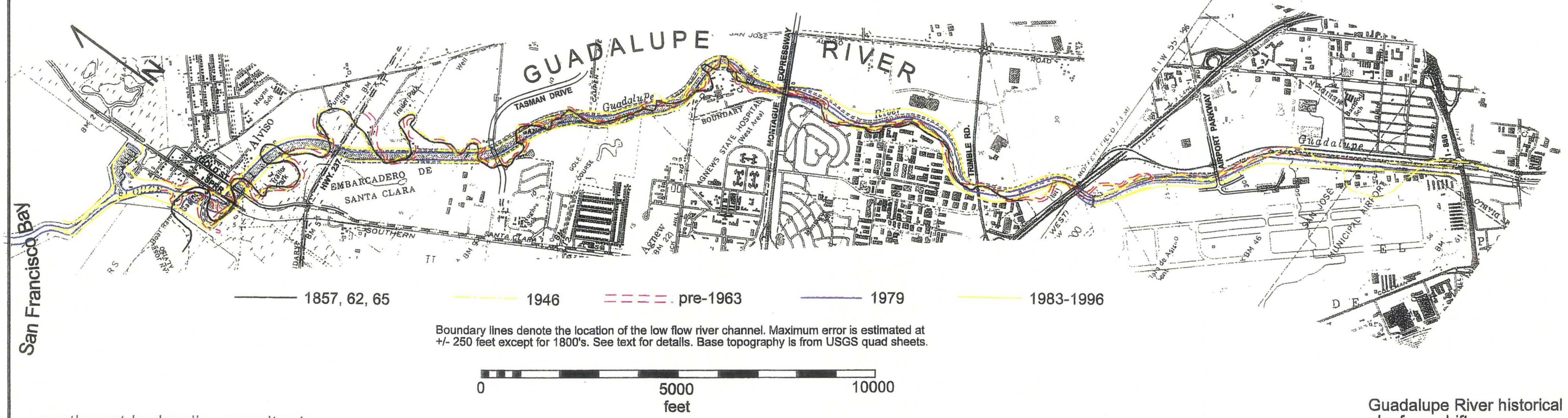
Map showing equal lines of land subsidence (in feet) in Santa Clara Valley from 1934 to 1967 (from Poland and Ireland, 1968) overlaid onto topographic mapping from the USGS. Guadalupe River is highlighted in blue.

Figure 6



Holocene sea level changes in South San Francisco Bay (from Atwater et al., 1977). Sea level has risen approximately 50 meters through the Holocene (last 10,000 years), lessening to a rate between 0.1 and 0.2 cm/year during the last 6,000 years.

Figure 7



Boundary lines denote the location of the low flow river channel. Maximum error is estimated at +/- 250 feet except for 1800's. See text for details. Base topography is from USGS quad sheets.

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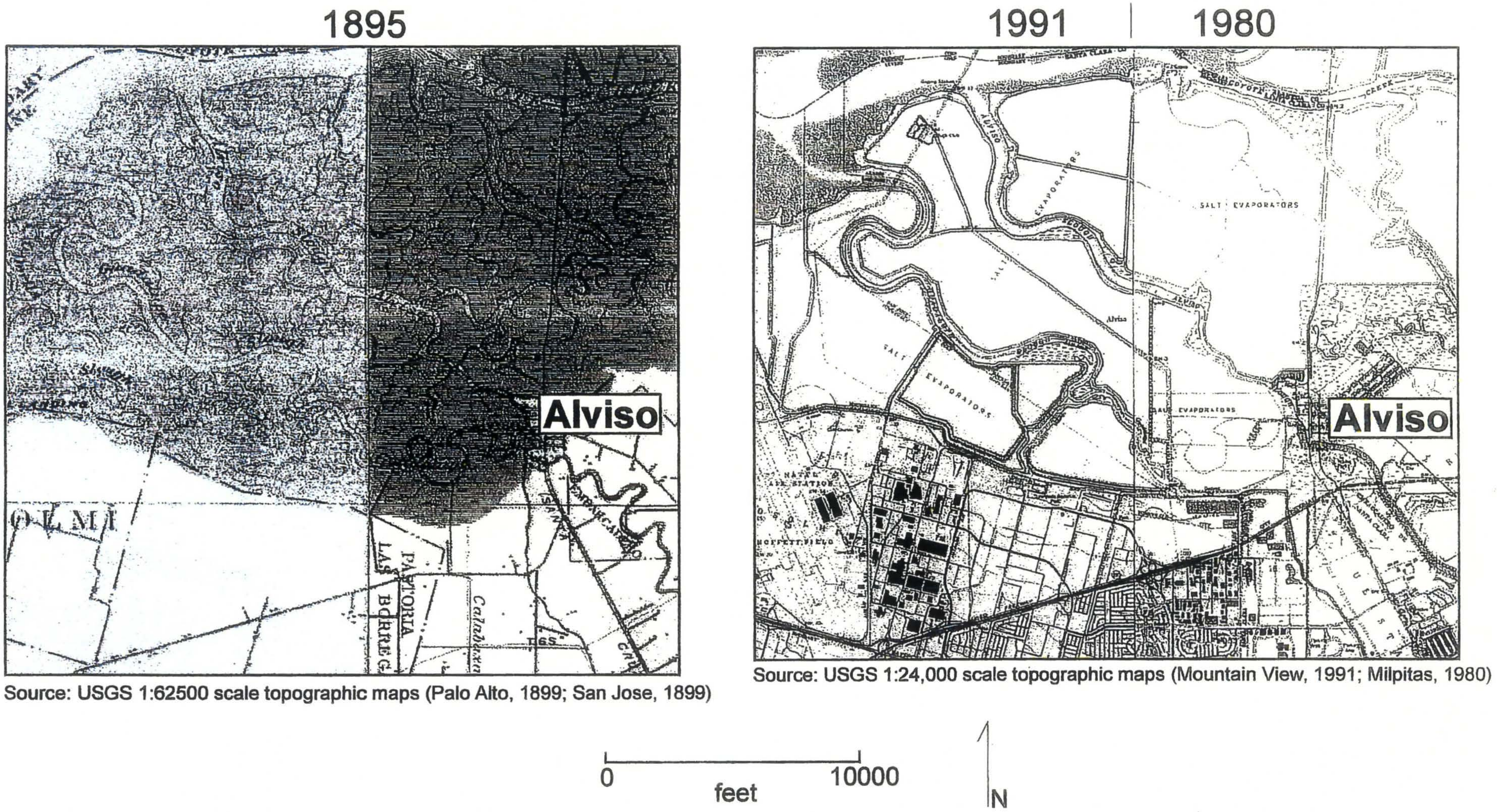
Guadalupe River historical planform shift map.

Figure 8

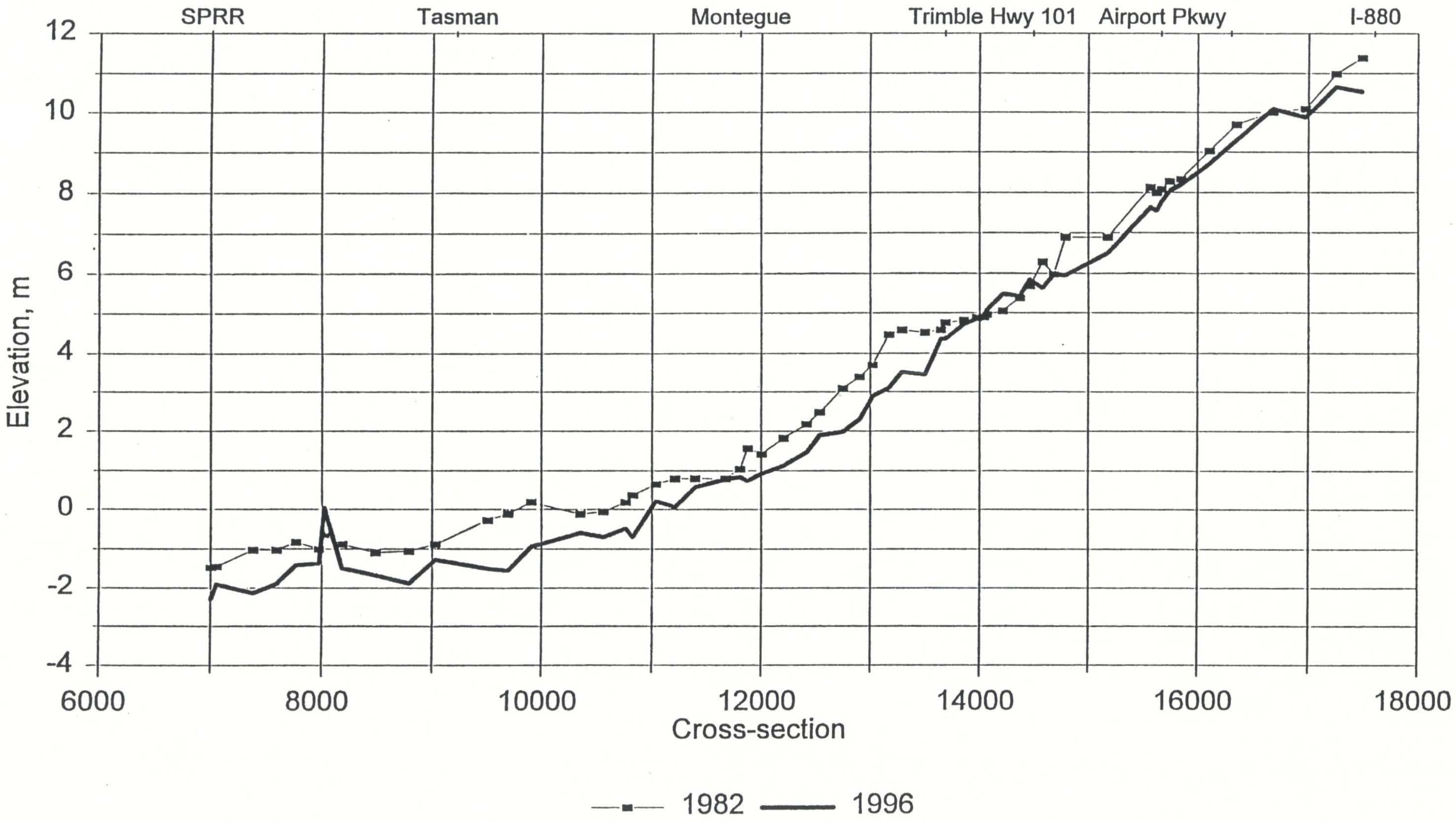


Source: USGS 1:62500 scale topographic maps (Palo Alto, 1899; San Jose, 1899)

Source: USGS 1:24,000 scale topographic maps (Mountain View, 1991; Milpitas, 1980)

Figure 9

Changes in Guadalupe River planform in the tidal marsh donwstream of Alviso from 1895 to 1980, 91. The original condition exhibits many distributary flow channels whereas presently all flows are carried through Alviso slough.



Guadalupe River invert profiles,UPRR to I-880 (from COE, 1989). All elevations set to NAD88.

Figure 10

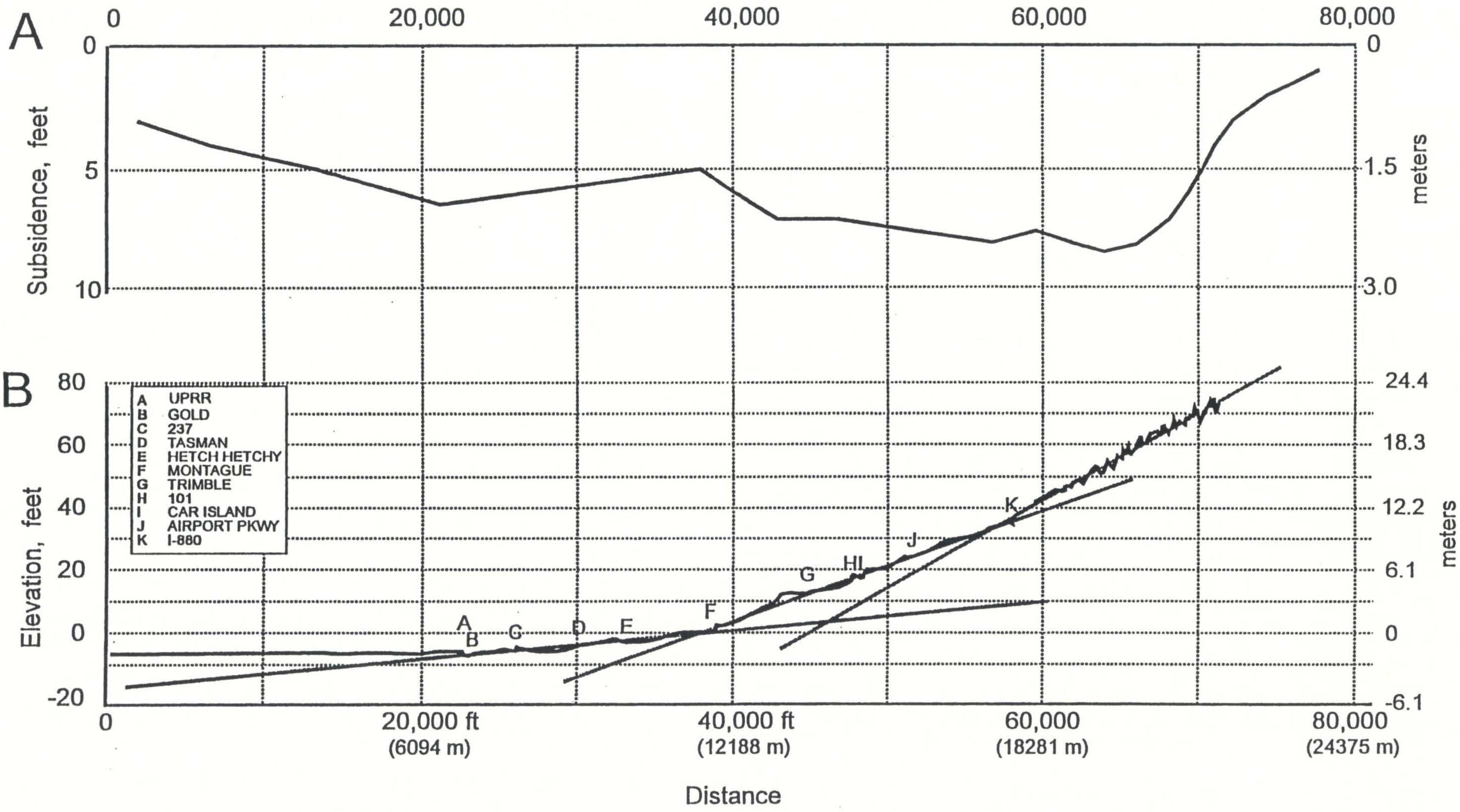
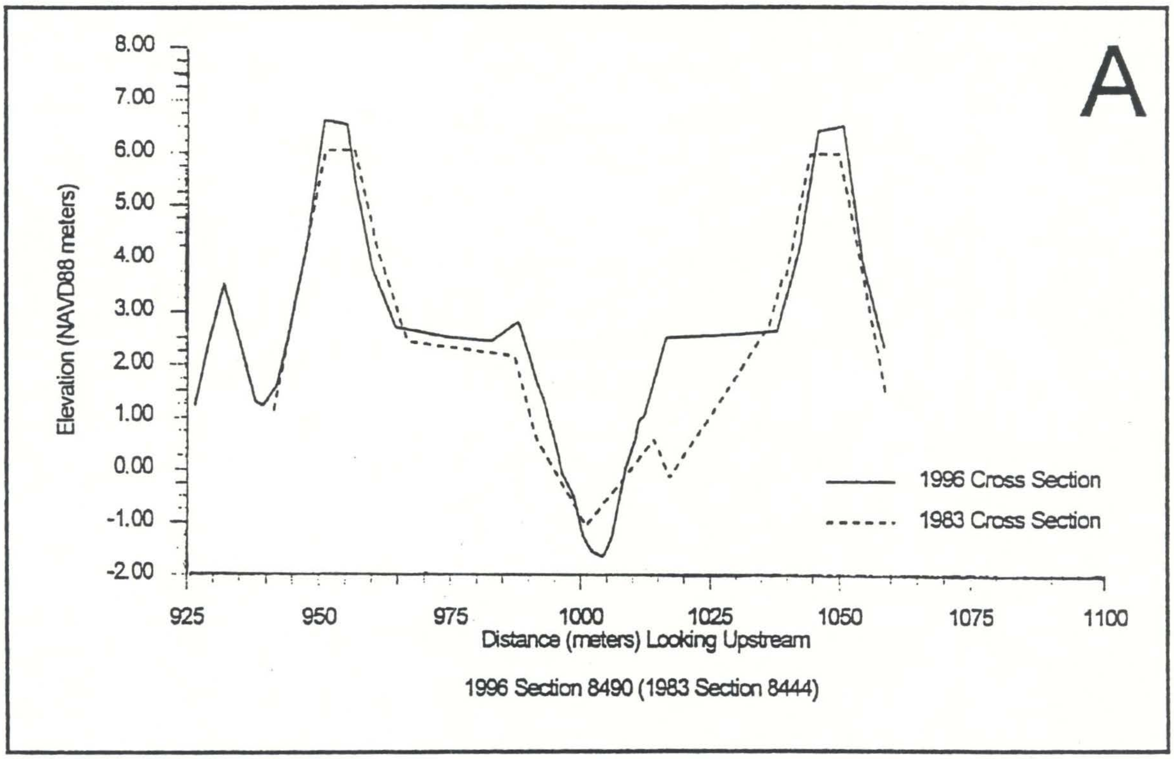
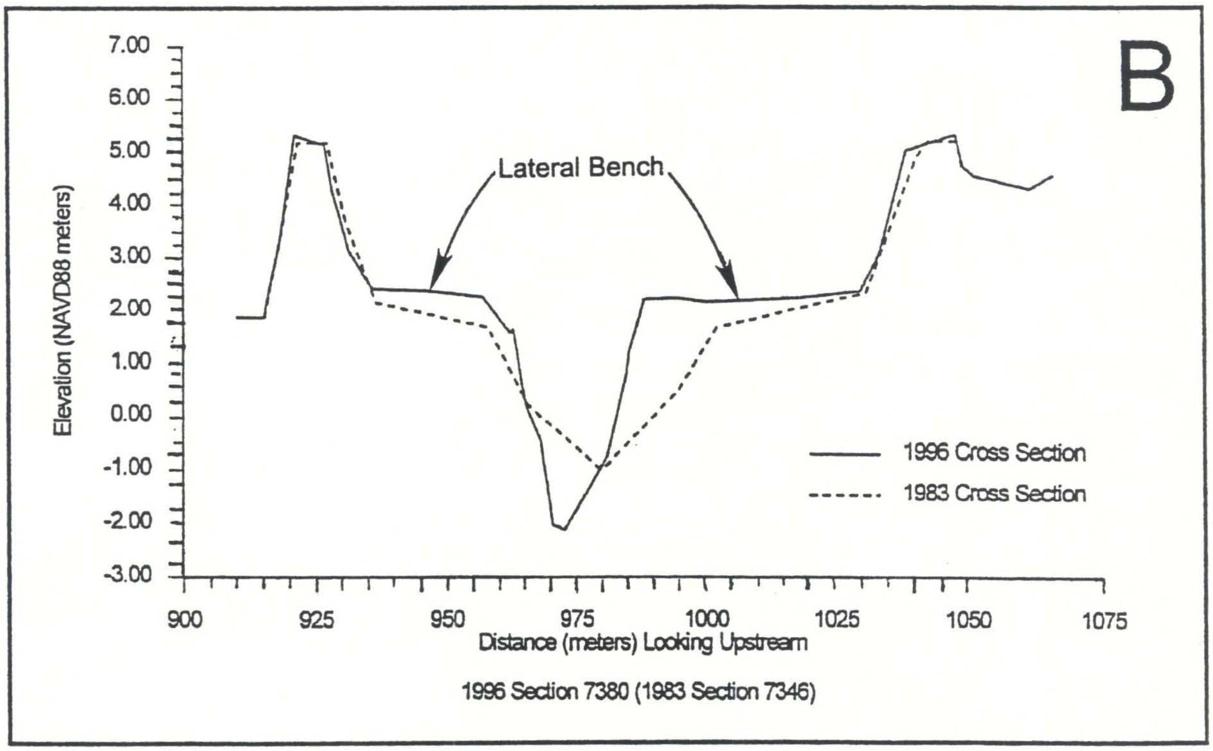


Figure 11

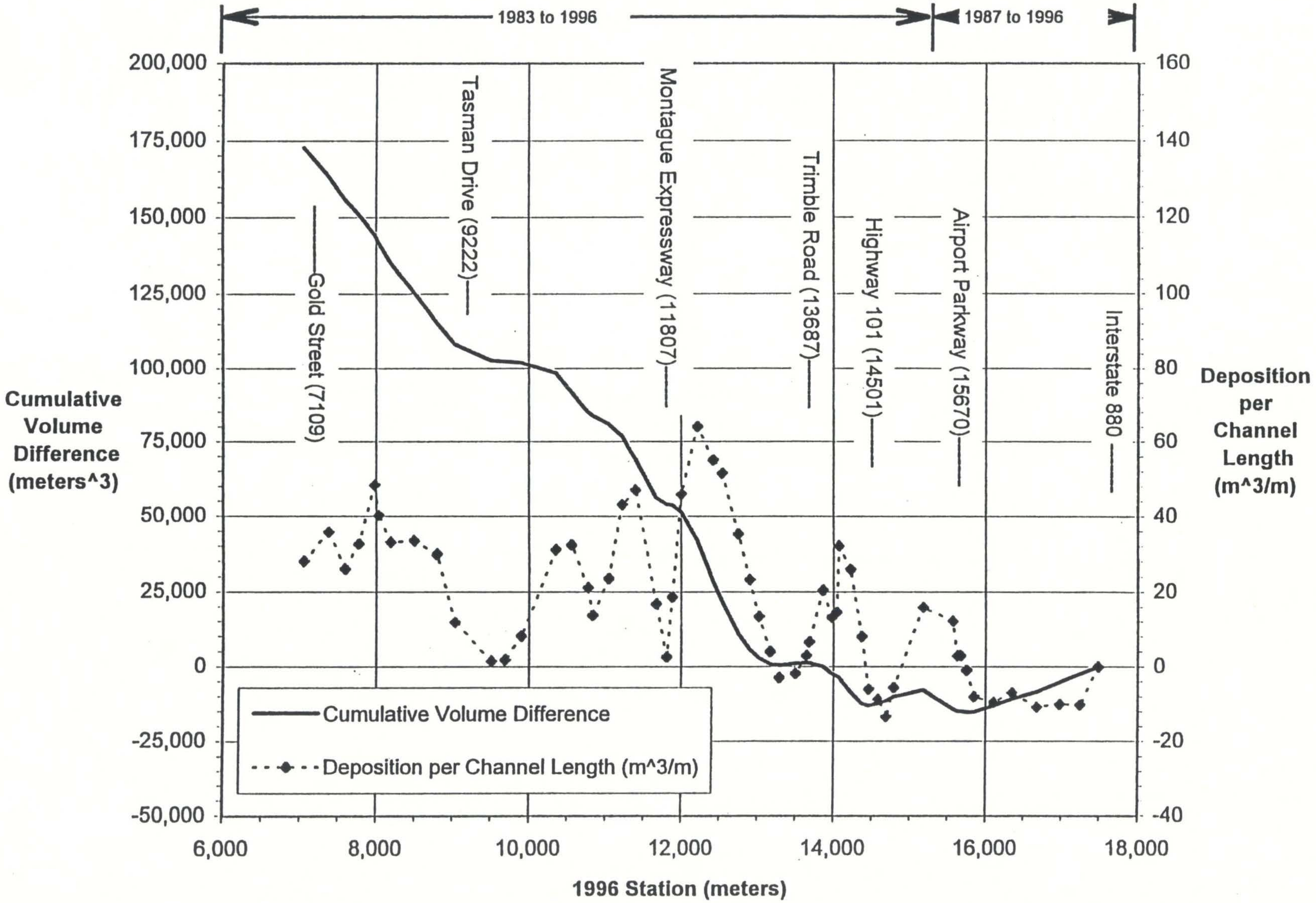
(A) Land subsidence along Guadalupe River from 1934 - 1967 (from Poland and Ireland, 1968) and (B) Guadalupe River profile (from COE, 1989). Lines identify regions of continuous channel slope. All data are set to NGVD 29.





Cross-sections from 1983 and 1996 show changes in channel geometry at (A) located 460 m upstream of Hwy 237 and (B) located 270 m upstream of Gold Street bridge. The low flow channel exhibits varying degrees of incision and narrowing resulting from sediment accumulation between the leveed banks.

Figure 12



Cumulative Volume Difference between 1983/1987 and 1996 Cross Sections

Figure 13

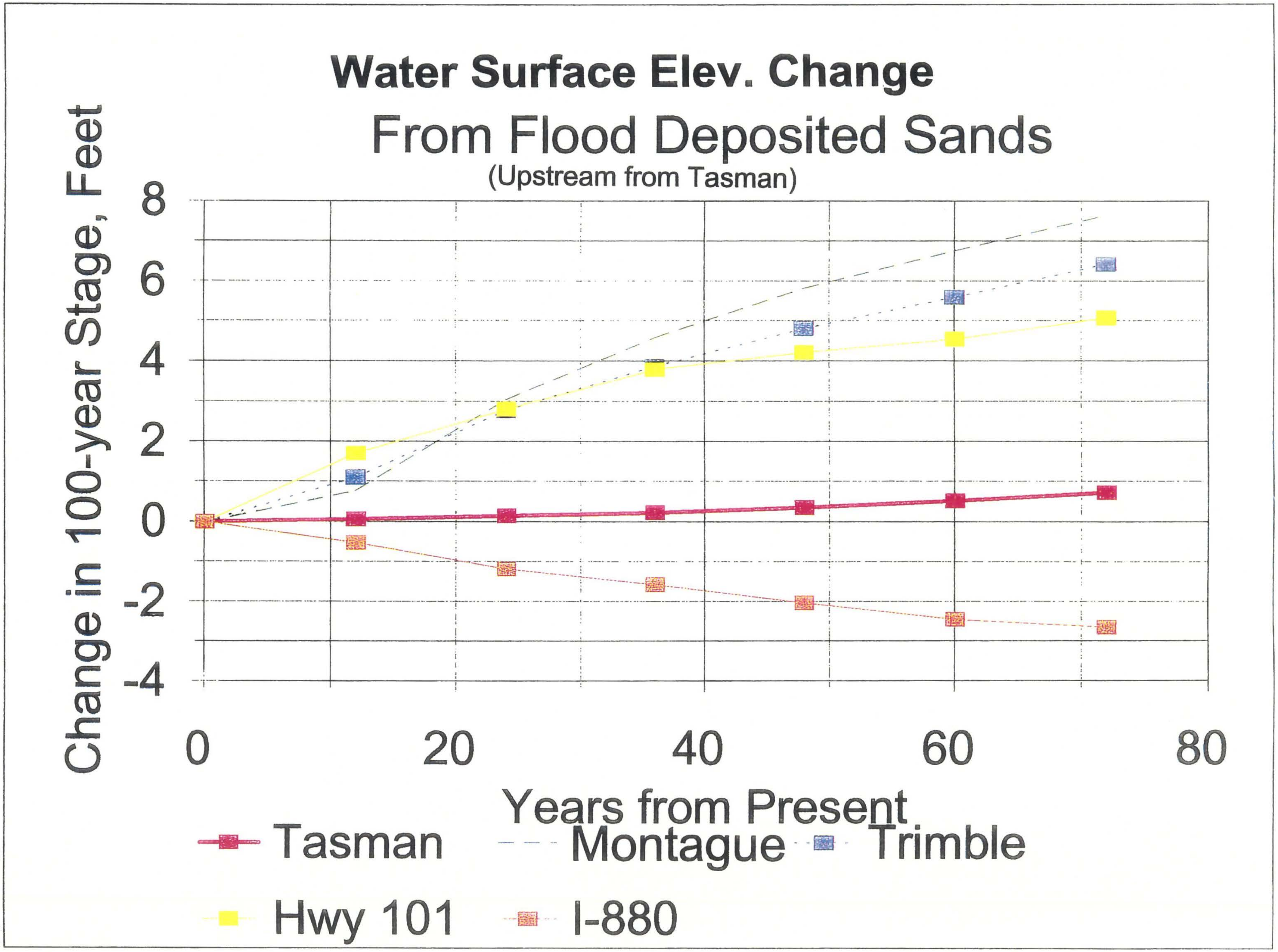


Figure 14

Water Surface Elev. Change
From Flood Deposited Sands
(Upstream from Tasman)
Change in 100-year Stage, Feet
8
6
4
2
0
-2
-4
0
20
40
60
80
Years from Present
Tasman
Montague
Trimble
Hwy 101
I-880

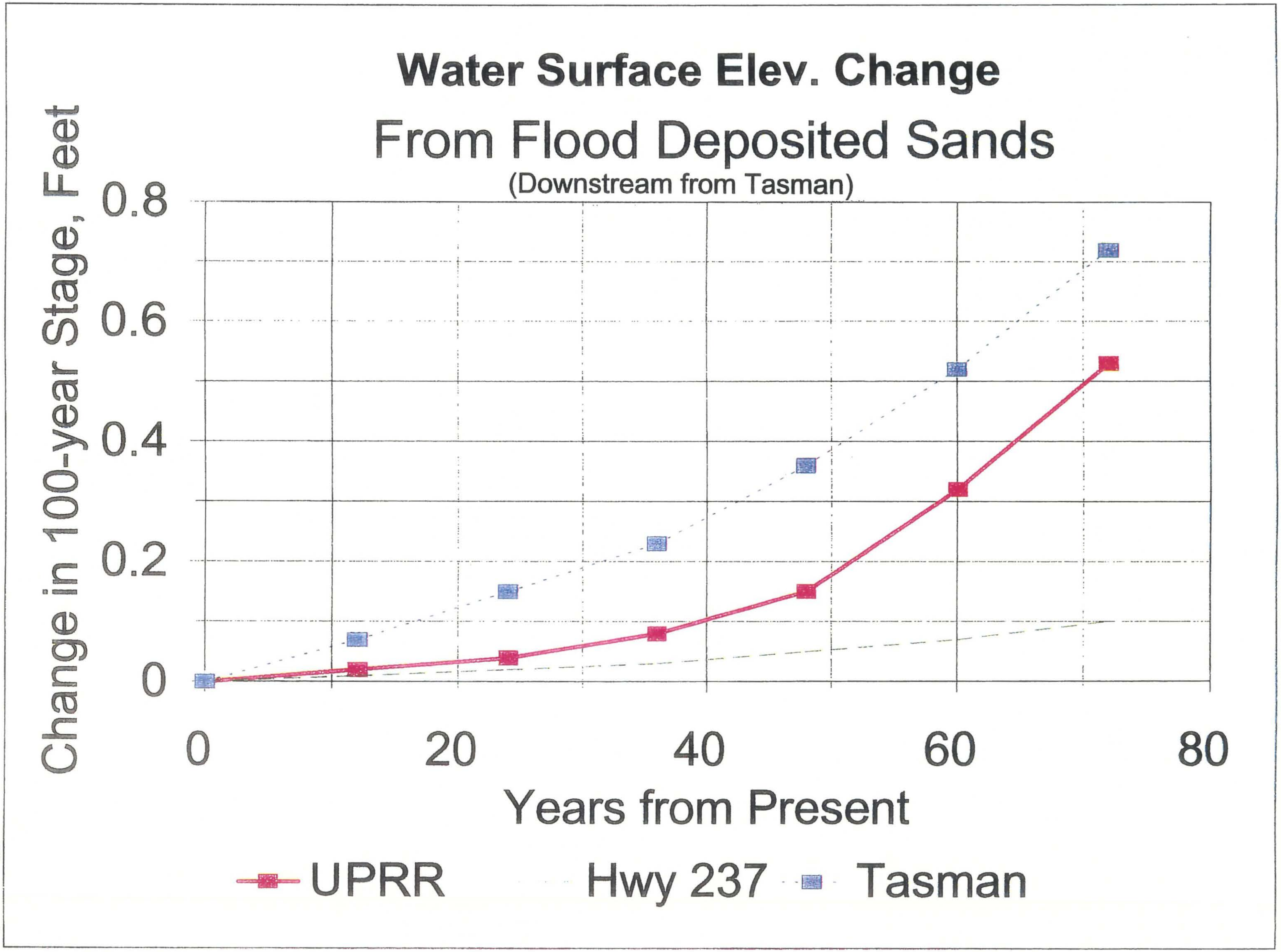

Water Surface Elev. Change
From Flood Deposited Sands
(Downstream from Tasman)
Change in 100-year Stage, Feet
0.8
0.6
0.4
0.2
0
0
20
40
60
80
Years from Present
UPRR
Hwy 237
Tasman

Figure 15

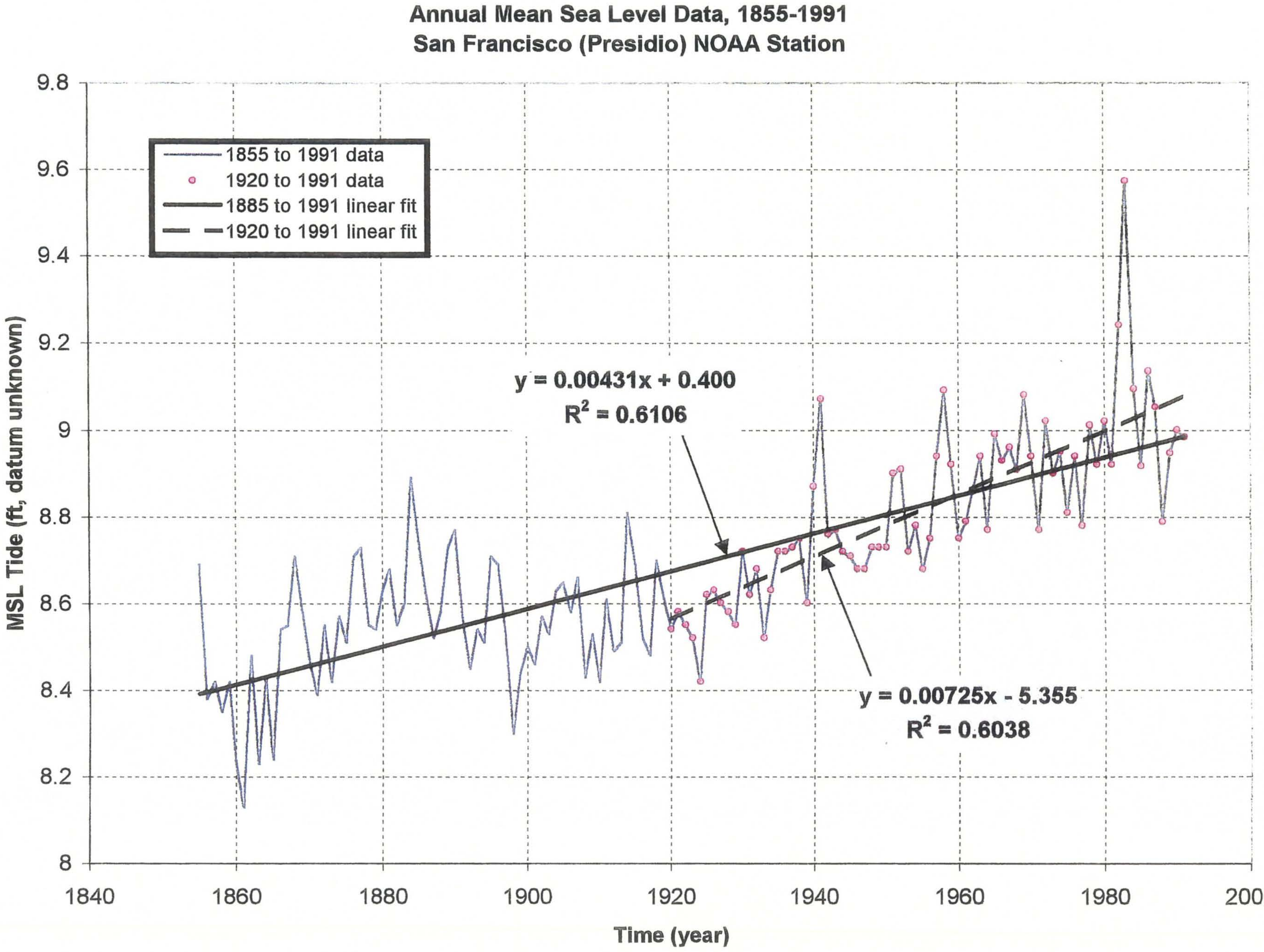

Annual Mean Sea Level Data, 1855-1991
San Francisco (Presidio) NOAA Station
9.8
9.6
9.4
9.2
9
8.8
8.6
8.4
8.2
8
MSL Tide (ft, datum unknown)
1840
1860
1880
1900
1920
1940
1960
1980
2000
Time (year)
1855 to 1991 data
1920 to 1991 data
1885 to 1991 linear fit
1920 to 1991 linear fit
y = 0.00431x + 0.400
R^2 = 0.6106
y = 0.00725x - 5.355
R^2 = 0.6038

Figure 16

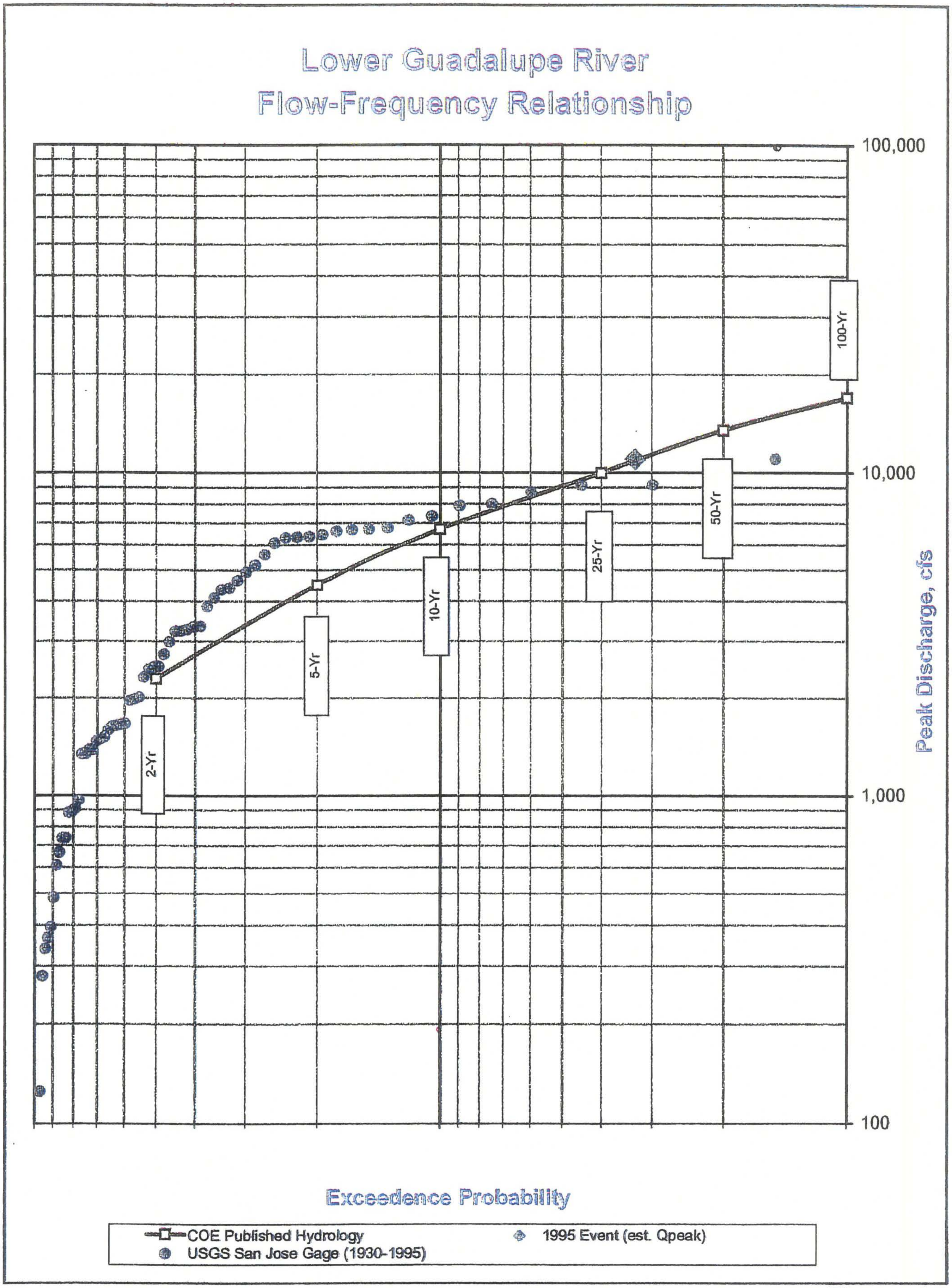
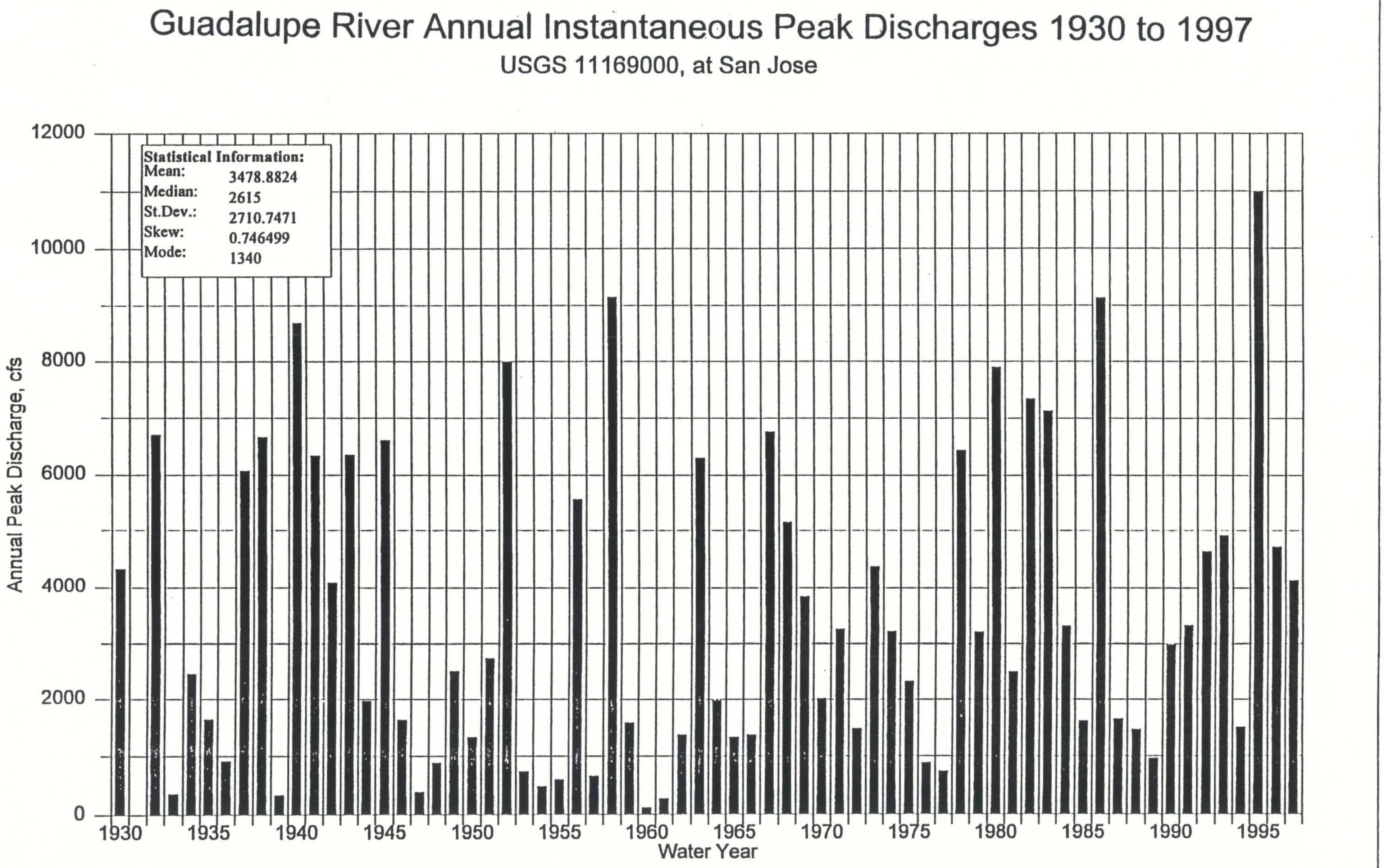

Lower Guadalupe River
Flow-Frequency Relationship
100,000
10,000
1,000
100
Peak Discharge, cfs
2-Yr
5-Yr
10-Yr
25-Yr
50-Yr
100-Yr
Exceedence Probability
COE Published Hydrology
1995 Event (est. Qpeak)
USGS San Jose Gage (1930-1995)

Figure 17

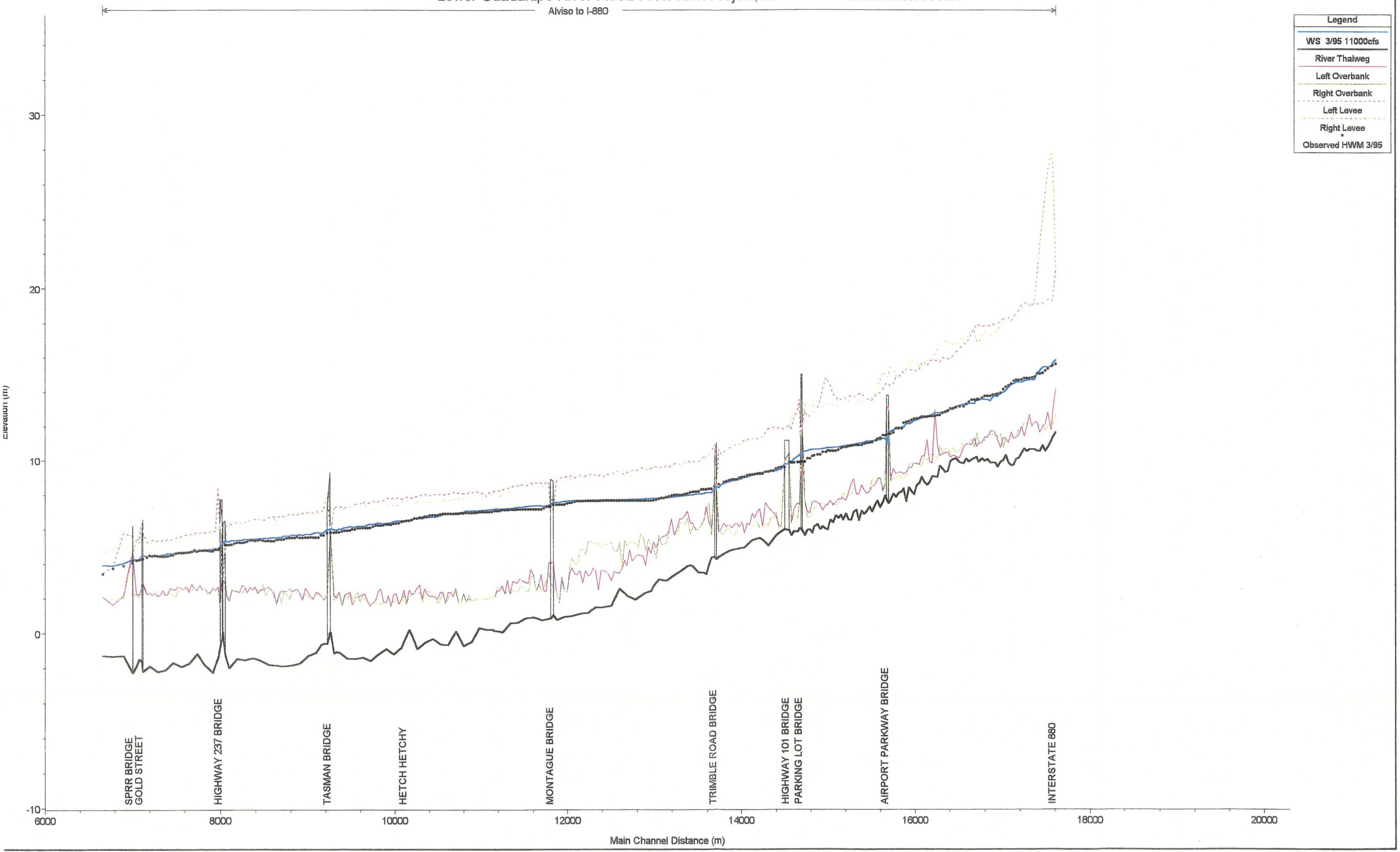
C:\Projects\50080\GUADPKS.WB2



Guadalupe River Annual Instantaneous Peak Discharges 1930 to 1997

USGS 11169000, at San Jose

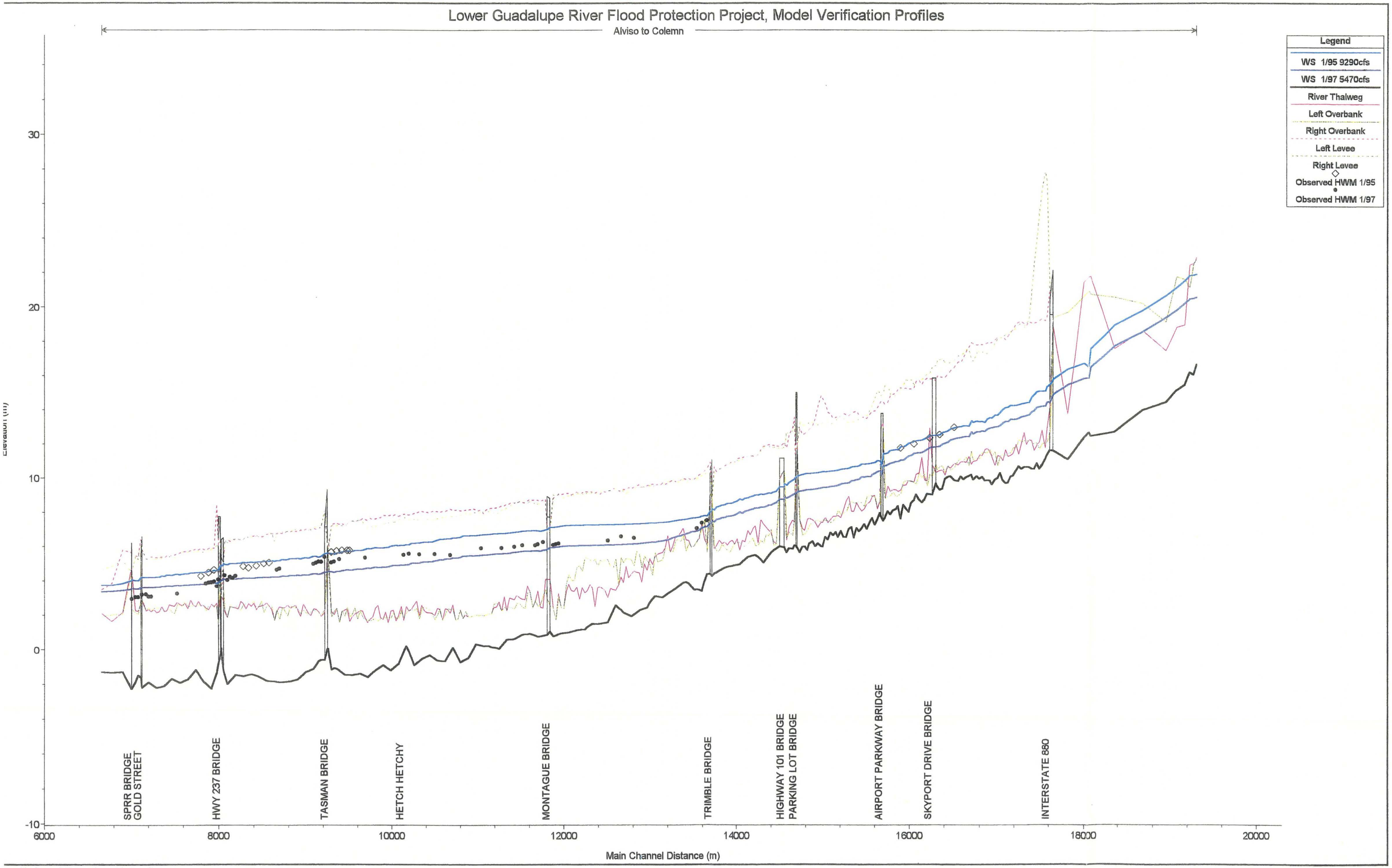
Figure 18



Lower Guadalupe River Flood Protection Project, March 1995 Calibration Profile

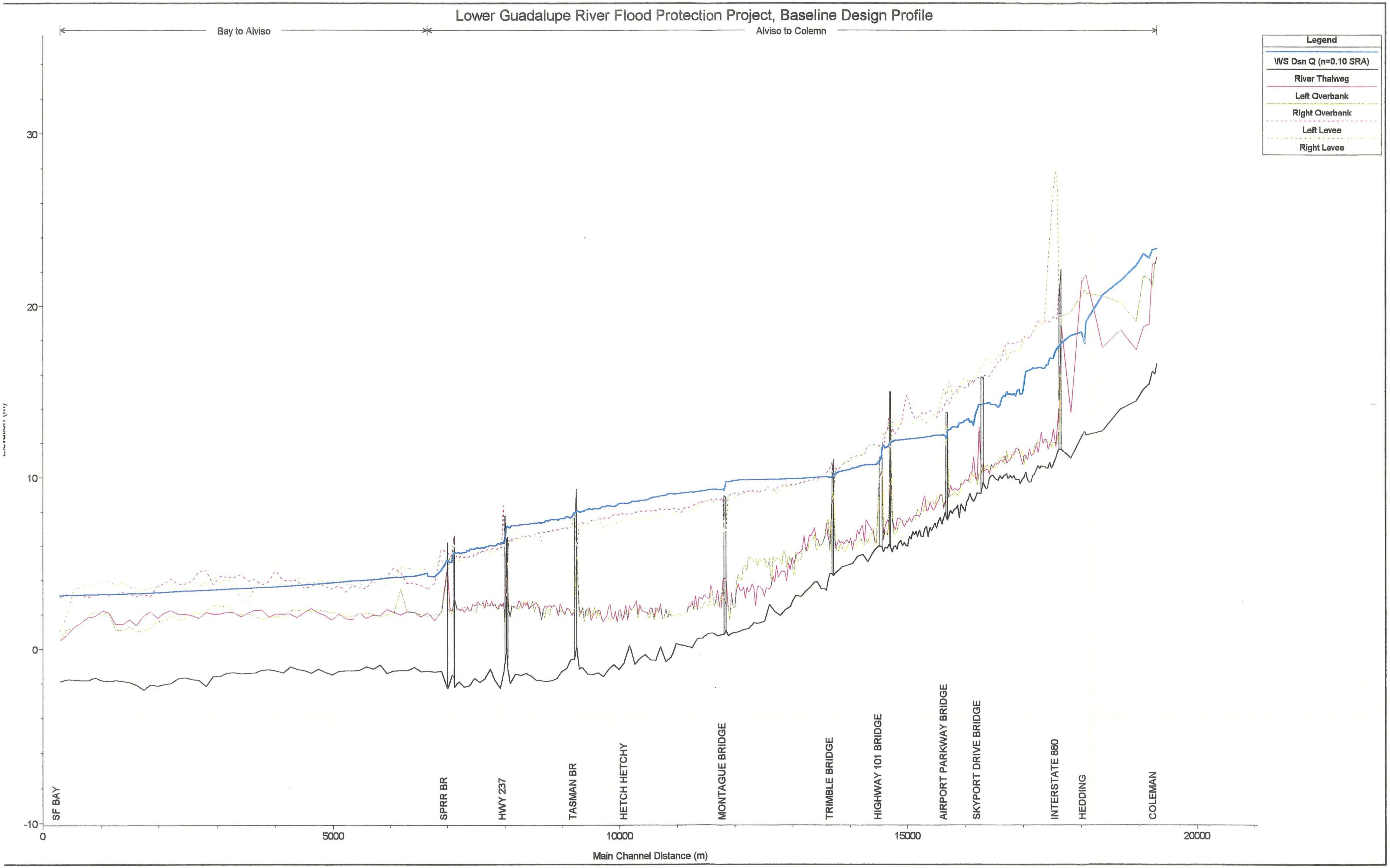
cm Horiz. = 400 m 1 cm Vert. = 2 m

Figure 19



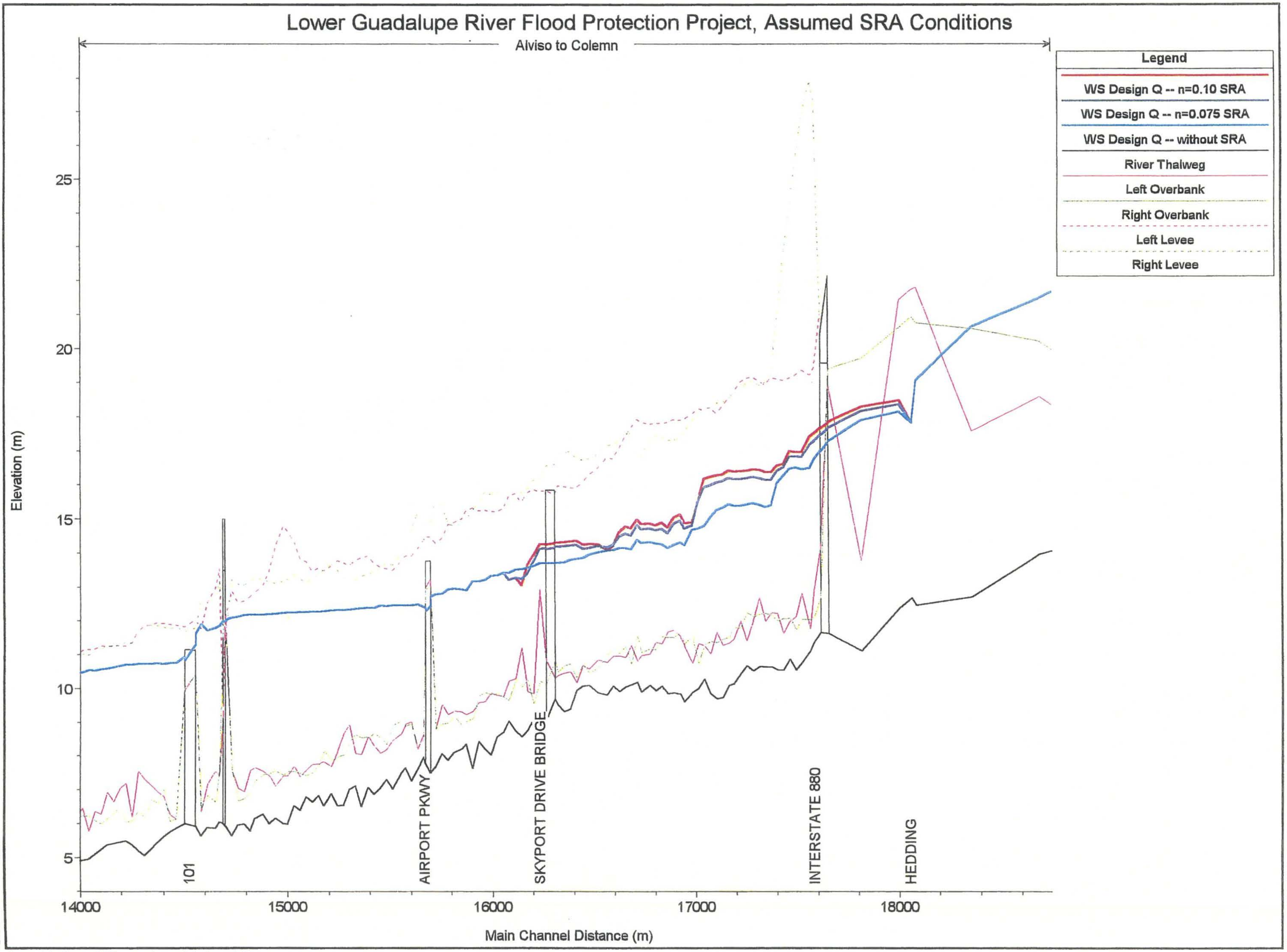
cm Horiz. = 400 m 1 cm Vert. = 2 m

Figure 20



cm Horiz. = 600 m 1 cm Vert. = 2 m

Figure 21



1 cm Horiz. = 250 m   1 cm Vert. = 1.5 m

Figure 22

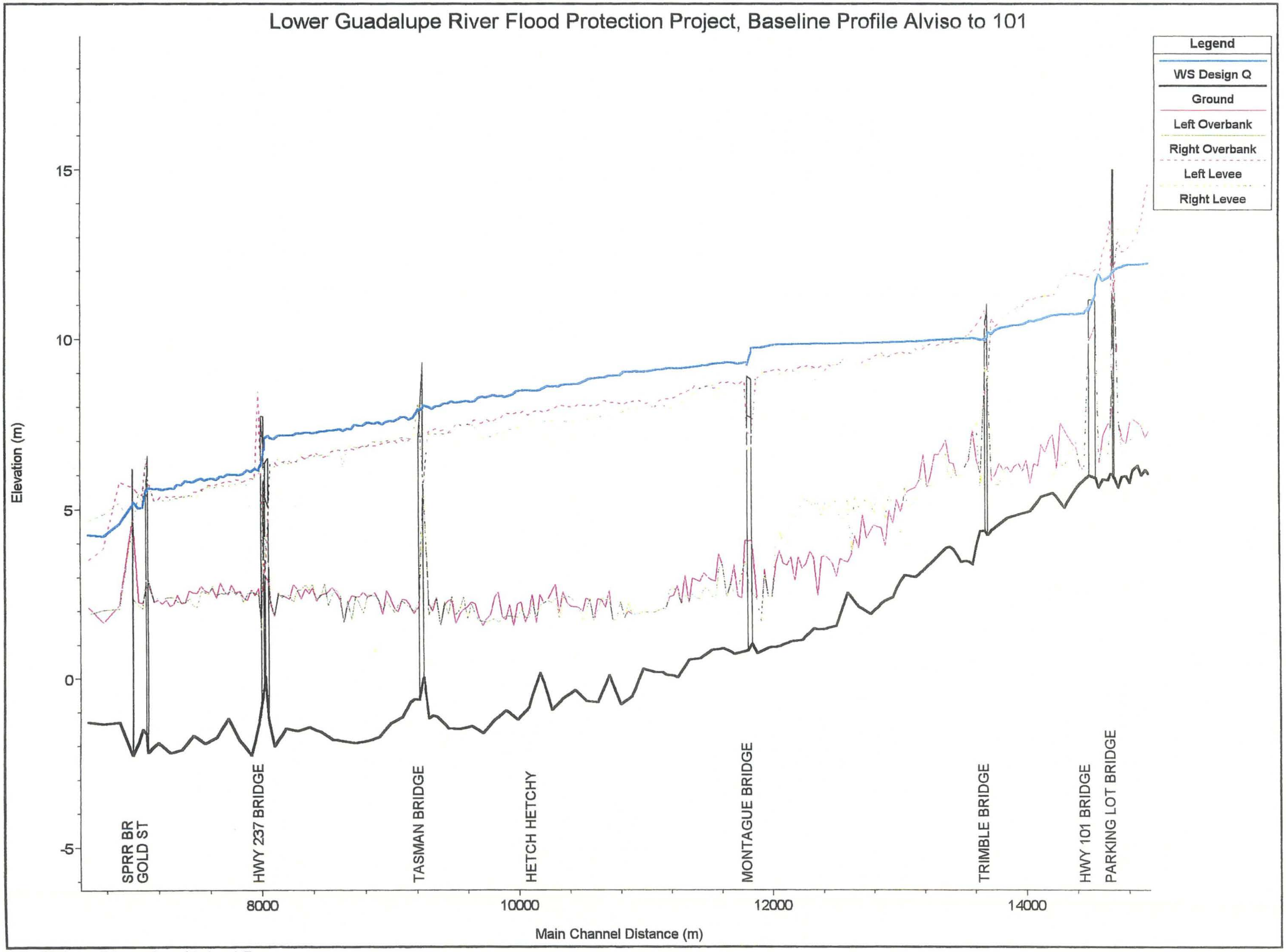
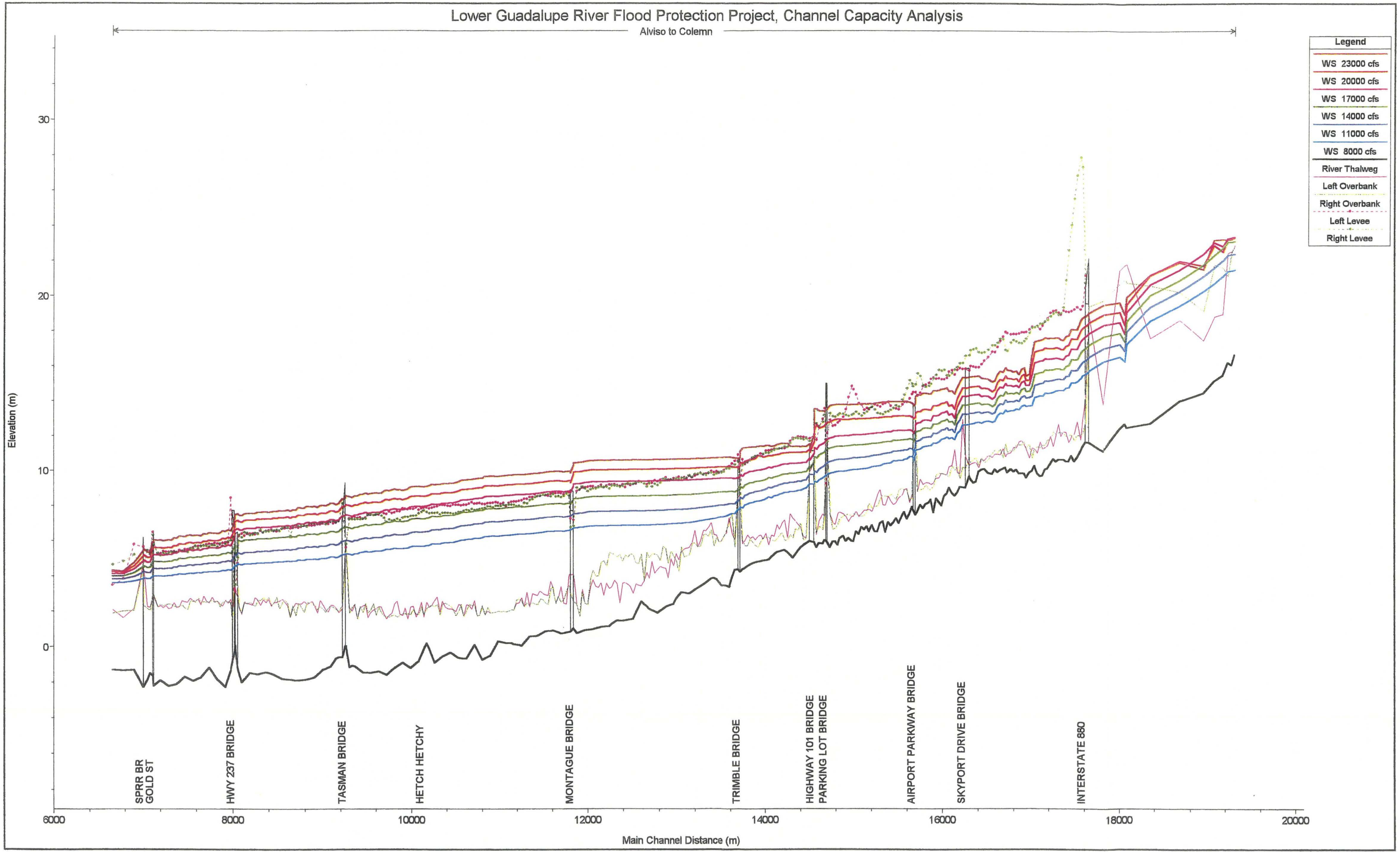


Figure 23

1 cm Horiz. = 400 m   1 cm Vert. = 1.5 m



1 cm Horiz. = 400 m 1 cm Vert. = 2 m

Figure 24